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FUNCTIONAL CLASSIFICATION OF GOUGE MATERIALS FROM SEAMS AND FAULTS IN RELATION TO STABILITY PROBLEMS IN UNDERGROUND OPENINGS

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13. ABSTRACT

The objective of the investigation is to develop a functional classification for gouge material as related to the stability problems in underground openings. To accomplish this objective, an extensive literature search has been undertaken along with on-site case studies, and laboratory investigations.

Background material is presented in Chapters III and IV by discussing classification schemes for rock masses and discontinuities and the characteristics of gouge material. Chapter IV lists the stability problems caused by fault underground and gives case histories. The testing program for this project and the cases studied are given in Chapter VI. Finally a Tentative Classification for Fault Gouge Material is presented in Chapter VII.

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TECHNICAL REPORT SUMMARY

This report summarizes some findings during the first year of a two-year effort for establishing a functional classification of gouge material from seams and faults as related to stability problems in underground openings.

The classification of rock masses in general and discontinuities in particular are discussed in part III. It is pointed out that discontinuities should be characterized using three different criteria, namely scale, based on aperture, persistence and typical spacing; character, based on smoothness, and the properties of filling material or coatings (if any); and strength and deformability, based on measured values obtained through testing. Five groups are suggested according to scale, seven groups according to character. It is emphasized that the character of a discontinuity is as important, sometimes more important than scale per se.

Some major characteristics of gouge material are discussed briefly in part IV. The gouge material will sometimes have the character of "crushed rock." Much more common is subsequent hydrothermal alterations and/or the deposition of hydrothermal products that are not associated with the petrography of the crushed rock or the wallrock. Thus, the mineralization in a fault is not necessarily a function of the components present prior to the hydrothermal action. Close to the surface, chemical weathering may have leached or changed the gouge. At depth, the mineralization may reflect low-grade metamorphism during faulting.

The properties of the gouge material in a fault or seam are only one of many factors that influence the stability of an underground opening in the vicinity of the fault or seam. Other factors are the size and shape of

the opening, method of excavation, method of support or reinforcement and lining, time ellapsed after excavation, the width of the fault or seam and its strike and dip in relation to the opening, adjacent faults or seams (if any), the frequency, orientation and character of joints in the adjacent rock, the competence of the wall rock, the in-situ stress in the gouge, and the water regime. These factors are discussed in part V, that also includes a summary of a number of slides in underground openings involving faults or seams.

Laboratory and field investigations carried out to date are reported in part VI. These investigations are directed at obtaining identification criteria for the different types of gouge material in terms of strength, deformability, swellability and slakeability.

A tentative classification of gouge material is suggested in part VII, based on 5 types of dominant material in the gouge. The potential behavior of the different types at the face and later is listed, and pertinent properties and parameters to be assessed or measured identified. Finally, the classification lists all factors that will influence the actual behavior.

During the second year, the classification will be adjusted as necessary, and "quantified" to the extent possible. The final report will also include a broad discussion on proper selection of excavation methods and support and lining systems through different types of gouge material.

I. PREFACE

This investigation was authorized by the Advanced Research Project Agency (ARPA) and was performed between February 12, 1971 and February 11, 1972 under Contract N. H0210023 between the Bureau of Mines acting as an agent for ARPA, and the Regents of the University of California, Berkeley. The report summarizes some findings made during the first year of a two-year effort for establishing a functional classification of gouge material from seams and faults as related to stability problems in underground openings. The second year, authorized under Contract No. H022022, will be used to continue the investigation of gouge material, and verify results obtained so far. Also, an investigation of proper construction procedure and support methods when tunnelling through fault zones will be carried out.

In addition to the authors of this report, several graduate students in Geotechnical Engineering at the University of California, Berkeley have contributed to the work. Mr. Charles E. Glass was a Research Assistant on the project during the Spring of 1971. Mr. Andre Kollbrunner assisted in the field investigations carried out at Straight Creek Tunnel, Colorado, during the summer of 1971 and in surveying literature in German. Mr. Jon Benoist has been working as a Research Assistant since January 1, 1972 assisting in the laboratory investigations.

The investigators acknowledge the valuable help given by many engineers and geologists in the course of the field investigations and the sampling of gouge material. This includes Mr. R. C. Hooper, District Engineer, Mr. John Post, Engineering Geologist, and other personnel with the Colorado Division of Highways at Straight Creek; personnel with the Straight Creek Contractors; with the Bureau of Mines in Denver; with the Bureau of Reclamation in Colorado and at the Auburn Dam Site in California; and Mr. B. Seegmiller, Research Engineer with the Anaconda Company.

The review of Mr. T. A. Lang, President, Leeds, Hill and Jewett, Consulting Engineers, San Francisco and Lecturer with the University of California, Berkeley is gratefully acknowledged.

During the work period covered by this report the project was monitored for the major part of the time by Mr. James J. Olson, and later by Mr. C. H. Von Hessert, both with the Bureau of Mines, Minneapolis Research Center. Their valuable suggestions and cooperation are appreciated.

II. INTRODUCTION

Stability of an underground opening can be defined in several ways. To geologists and engineers that are used to working in geological environments where the major parts of the works after careful scaling can be left unsupported during the construction period as well as during operation, any rock mass behavior that requires reinforcement, support or lining may be regarded as a stability problem. To colleagues excavating underground openings in geological enviornments that require continuous use of reinforcement or support and lining "stability problems" are often associated only with those parts of the work where "standard" construction procedure and method of reinforcement or support and lining is inadequate, and special measures therefore needed. In either case, a major objective is to assess the stability problems correctly - as far as possible at the planning stage - and choose methods for excavation and reinforcement or support that will bring the opening safely and as economical as possible through the problem areas.

Tunnels and other underground openings are fundamentally different than most civil engineering structures in several respects. Through careful investigation at the planning stage of an underground opening, it is often possible to improve stability and reduce cost by finding the most favorable placing, orientation, shape of opening and excavation procedure. However, it is in general not possible to choose a particularly favorable rock mass in which to place the opening. As opposed to most other civil engineering works it is therefore not possible to "choose" the most suitable materials.

Secondly, the present knowledge of the structural properties and behavior of rock masses is quite limited when compared to the knowledge of most other civil engineering materials, e.g., concrete and steel. There are several facets to this problem. Rock masses are so variable in nature that the chance for ever finding a common set of parameters and a common set of constitutive equations valid for all rock masses is quite remote. Simplified engineering-geological classifications as well as sophisticated mathematical formulations have in many instances proven to be valuable tools in assessing rock mass behavior. However, they are often both in literature as well as in engineering practice given a general validity although they may often be highly inadequate both from the point of view of restrictive assumptions, and from the point of view of the variability of rock masses. Therefore, they are often more misleading than helpful, giving a fault feeling of adequate design procedures.

The problem of ascribing the correct properties to rock masses are further compounded in many instances by changes in properties due to environmental changes such as drying and rewetting.

Thirdly, almost unique to underground openings is that they "puncture" existing stress fields. Later, the significance of this will be discussed in more detail. Here it should only be emphasized that increased stresses do not necessarily mean increased instability. While for most other civil engineering works, structures will be designed to keep stresses in the different components below given levels, problems in tunnels have often been associated with insufficient in-situ stress, causing instability.

Finally, underground openings are different from most other civil engineering works in that the "loads" that will develop are often difficult to define, not least because they in a given rock mass are dependent on the size and shape of the opening, the excavation procedure, and the method of reinforcement, support or lining. As will be discussed later, time is in this connection a most important parameter.

However poor and erratic knowledge of rock mass behavior is at present compared with the knowledge for most other civil engineering materials and structures, "quantitative" decisions have to be made regarding excavation procedure as well as type and "amount" of reinforcement or support and lining. The choice of correct methods and procedures for driving openings through faults and seams has proven to be among the most difficult assessments. One of the factors that strongly influences the outcome are the properties and behavior of the gouge materials in these discontinuities. The purpose of the ongoing effort is to develop a functional classification system for such materials, functional in the sense that it can be readily understood and used and at the same time lead to a correct assessment of methods and procedures needed to insure a safe and economical opening both during construction period and for the lifetime of the project.

III. CLASSIFICATION OF ROCK MASSES AND DISCONTINUITIES

One of the first efforts to classify rock masses in terms of the rock loads that may be mobilized against support in an underground opening was made by Bierbaumer (1913) in Leipzig. Between the World Wars the development in Central Europe was led by Professor Stini; his textbook on tunnel geology (1950) includes a classification of rock masses, and a very detailed and well documented treatise of adverse conditions in tunnelling. More recently, classification systems have been suggested by Rabcewicz (1957) and Lauffer (1958).

Rabcewicz's work is known in the United States through several papers in Water Power describing the so-called Austrian tunneling method. The basic principle of this method is to install reinforcement or support as fast as possible after excavation in order to secure that the inward movement of the rock mass is kept at the minimum. The method has sometimes been assumed to be synonomous with the use of shotcrete, while it in general includes any measure that can rapidly be applied close to the face.

The very important contribution by Lauffer is his emphasis of the importance of "stand-up time" relating for different classes of rock masses the time an opening can stand unsupported to the "active span." The active span is the width of the tunnel or the distance from support to the face in case this is less than the width of the tunnel. Other major contributions from Central Europe includes the work by Zaruba and Mencl (1961), Kastner (1962) and Müller (1963).

In Scandinavia, where extensive use of underground openings has been made during the last thrity years, classification systems that either fully or in part refers to tunnelling conditions have in Sweden been suggested by Bergmann (1965); Hansagi (1965), and Hagerman (1965). In Norway, Selmer-Olsen

(1964) and co-workers have used a more "individual" approach, assessing as detailed as possible the extent of particular problems to be met in each underground opening, and ascribing in each case the remedial measures that seem necessary and sufficient. This approach has proven to be the most rewarding both technically and economically under the generally good tunnelling conditions in Norway.

In the United States, the classification system proposed by Terzaghi (1946) has for the last 25 years been completely dominant, either in its original form or with some modifications. More recently, a classification based on modified core recovery (RQD) suggested by Professor Deere of the University of Illinois, has found considerable use.

Each one of the classification systems available has tried to meet the following requirements:

(1) being simple and meaningful in terms.

(2) being based on parameters that can be measured or assessed rapidly and inexpensively.

(3) being exact enough to yield quantitative data that can readily be applied in engineering design.

Most of them have proven to be of great value in geological engineering when carefully used, considering the conditions that are specific to each individual site. On the other hand, most of them are continuously misused because the premises for and assumptions made in developing the classification systems have not been carefully studied by the users, and because they have been given a validity for "quantification" of rock mass behavior that is far more general than was intended by their authors.

Most of these classification systems include rock conditions typical for fault zones. For example, Terzaghi (1946) has four classes, namely:

"Completely Crushed but Chemically Intact", "Squeezing Ground, Moderate Depth", "Squeezing Ground, Great Depth", and "Swelling Rock."

Before a further discussion is given on faults, it seems advantageous to discuss in some detail discontinuities in rock masses in general.

Such discontinuities are surfaces of rupture resulting from failure under natural forces. In addition, fractures may be induced during blasting and due to stress relief. Such fractures are not considered here, nor is renewed rupture that may take place due to an earthquake during the lifetime of an underground opening transversing an active fault.

Discontinuities are formed through failure in extension/tension, in shear, or in more complex failure through a combination of both. Rupture surfaces from extension are characteristically rough and clean with little detritus. If such surfaces are pressed back together and a direct shear test performed, the resulting load-deformation curve would display a peak rising well above the residual strength which would only be reached at large values of displacement.

Simple surfaces of shearing are characteristically smooth with considerable detritus. A direct shear test along the surface would not yield as great a contrast between peak and residual strength as in the case of extension fracture. Furthermore, discontinuities formed in shear are in general much more susceptible to alteration than are those formed in extension. Normally, such alteration leads to a lowering of the strength and to other disadvantageous conditions for construction.

As to a more detailed classification of discontinuities, there are not any distinct and generally accepted rules or nomenclature for engineering purposes. However it seems reasonable to make use of three criteria for classification:

SCALE, based on aperture, persistence and typical spacing.

CHARACTER, based on smoothness and the properties of filling material or coatings if any.

STRENGTH AND DEFORMABILITY, based on measured values obtained in the laboratory and field testing.

A. Scale

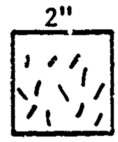
Figure 1 gives a breakdown in five classes according to scale. Microfissures are included in the table although they are usually considered a rock type characteristic. They are flaws in the rock type rather than genuine discontinuities.

Bedding plane and foliation partings are separations parallel to a mineralogically defined weakness direction in the rock. Since they are most often tight and rough, they may not be of major concern unless their orientation alone or in combination with other discontinuities is unfavorable in relation to the orientation and shape of the underground opening. For instance, close horizontal partings in a tunnel roof may necessitate a rather pointed cross-sectional profile and bolting of the flanks - partings, and in particular foliation partings are often developed to joints. They should then be classified as such.

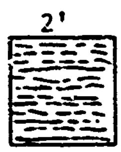
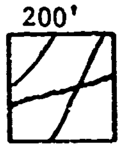
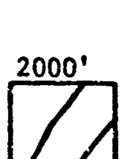
Joints and seams are the discontinuities described by Selmer-Olsen (1964) as the "detailed discontinuity pattern." The term "seam" indicates a minor, often clay-filled zone with a thickness of a few inches. When occurring as a weak clay zone in a sedimentary sequence, a seam can be considerably thicker. Otherwise, seams may represent very minor faults or altered zones along joints, dikes, beds or foliation.

Joints have a great variety in aperture, persistence, frequency and character. Many geologists will reserve the term for typical extension discontinuities, without any perceptable movement along the joint. Here, however, joint is used as a scale term and therefore also includes minor shear ruptures.

Classification of Discontinuities According to Scale

Name		Typical Spacing
Microfissures		0.1 in.

(Usually considered a rock type characteristic)

Name		Typical Spacing
Bedding and foliation partings		1" to several inches
Joints (A) and Seams (B)		A. Several inches to several feet B. Tenths of feet
Minor faults		Tenths to hundreds of feet
Major faults		Hundreds to thousands of feet

The aperture may vary from tight to a fraction of an inch, the persistance from less than a foot to hundreds of feet; exceptionally a mile (tectonic sheet joints).

A typical feature of joints (and also often of seams) is that they occur in sets. In each set, the joints have approximately the same strike and dip, and usually the same character at least over a certain area, e.g. a few square miles. The joint system of a rock mass is usually made up of three or four sets. One or two sets can be completely dominating in frequency. A few "wild" joints and seams not belonging to the joint sets are not uncommon, and it also happens that the jointing is mosaic, i.e. there are apparently no distinct sets present in the joint system. Granitic rock masses may have a mosaic joint pattern but will more often have a joint system that yields roughly cubic rock blocks. Regional metamorphic rocks often have a system yielding rhombic rock blocks. Thick lava beds often have columnar jointing yielding hexogonal, prismatic rock blocks. In folds, sets often develop parallel to bedding, radial along the fold axis, and perpendicular to the fold axis.

Sheet joints or lift joints are spalling features resulting from extension failure due to unloading (e.g. erosion of overlying masses) or very high residual stresses due to other causes. They are best developed in massive rock masses, e.g. many places in the Sierra Nevada granodiorite batholite. They run almost parallel to the surface, and rapidly decrease in frequency in depth.

A particular kind of joints often found in regional metamorphic rock masses are the so-called "fiederspalten" or "feather cracks." They are tensile ruptures and they vary in size from inches to hundreds of feet.

Faults are the major rupture zones encountered. Minor faults range in width from several inches to several feet; major faults from several to hundreds, occasionally thousands of feet. In geology, great attention is given to the relative displacement along fault zones, and the faults are basically classified as normal, reverse, lateral and thrust faults. From an engineering point of view, this classification may not be of great importance. More important is the realization that most fault zones are the result of numerous ruptures throughout geological time, and that they quite often have other parallel discontinuities that decrease in frequency and thickness when moving away from the central zone.

B. Character

It is just as important - often more important - to classify discontinuities according to character as it is to note their scale parameters. For partings and joints, the first distinction can be made between rough versus smooth surfaces. The latter will most often be much more severed from an engineering point of view than a rough discontinuity. Secondly, it is important to distinguish between major types of coating and/or filling materials. It seems relevant to discuss seven groups.

(1) Joints, seams and sometimes even minor faults may be healed through percipitation from solutions of quartz or calcite. In this instance, the discontinuity may be "welded" together. Such discontinuities may, however, have broken up again, forming new surfaces. Also, it should be emphasized that quartz and calcite may well be present in a discontinuity without healing it.

(2) Clean discontinuities i.e., without fillings or coatings. Many of the rough joints or partings have this favorable character. Close to the surface, however, it is imperative not to confuse clean discontinuities with "empty" discontinuities where filling material has been leached and washed

away due to surface weathering.

(3) Calcite fillings may, particularly when they are porous or flaky, dissolve during the lifetime of an underground opening. Their contribution to the strength of the rock mass will then, of course, disappear. This is a long time stability (and sometimes fluid flow) problem that can easily be overlooked during design and construction. Gypsum fillings may behave the same way.

(4) Coatings or fillings of clorite, talc and graphite give very slippery, i.e. low strength, joint, seams or faults, in particular when wet.

(5) Inactive clay material in seams and faults naturally represents a very weak material that may squeeze or be washed out.

(6) Swelling clay may cause serious problems through free swell and consequent loss of strength, or through considerable swelling pressure when confined.

(7) Material that has been altered to a more cohesionless material (sand-like) may run or flow into the tunnel immediately following excavation.

It should only be emphasized again that the character of the discontinuities is at least as important as frequency from an engineering point of view. Thus, joint frequency per se is not sufficient basis for evaluating the behavior of a jointed rock mass. A much more complete classification, and at the same time, short and meaningful, would be for example "sleek, chloritic, persistant joints spaced 2 ft - 4 ft", or "1 in - 2 in seam carrying highly consolidated swelling clay and porous calcite."

C. Strength and Deformability

The shear strength and deformation characteristics of joints are presently being studied by R. E. Goodman and co-workers under Contract H0210020. Their investigation includes variations in normal stress, roughness, thickness of filling, shear rate, cleftwater pressure and back pressure.

Through the development of the so-called "joint element" (Goodman, Taylor, and Brekke, 1968) it became feasible by finite element analysis to study the behavior of jointed rock masses. Originally, the element behavior was defined as having no strength in tension, a shear strength (cohesion and friction), a normal stiffness, and a shear stiffness, and a classification system was suggested on the basis of these parameters. Later, dilatancy has been included (Dubois and Goodman, 1971).

The joint element is one-dimensional. As such, it is quite adequate for modelling the detailed discontinuity pattern in a "blocky" rock mass. For a stability analysis of a tunnel through a fault zone, however, it seems from a scale point of view more realistic to model the zone as a two-dimensional, preferably three-dimensional continuum media. The major problem is to decide on the correct constituative equations for the behavior of the gouge material. Hopefully, the shear test data and the field data when analyzed can contribute to some extent to solve this problem.

IV. CHARACTERISTICS OF GOUGE MATERIAL

Gouge material is very seldom uniform, reflecting that numerous movements have taken place within the fault zone. Blocks, or even plates, of intact rock may "float" in a basic matrix of soft material, that again may have bands or seams of hard material, e.g. quartz or calcite. Thus, fault gouge is normally a very complex material both in regard to mineralization and in regard to physical properties.

The gouge material will sometimes have the character of unaltered "crushed rock." Much more common is subsequent hydrothermal alterations and/or the deposition of hydrothermal products that are not associated with the petrography of the crushed rock or the wall rock. This is most important since it implies that the mineralization in faults and seams is not necessarily a function of the "original" composition of the rock mass in the faulted area. Therefore, a given mineralization, e.g. of specific clay minerals, cannot be excluded on the basis of the minerals present prior to the hydrothermal action. On the other hand, the susceptibility for alteration of the "original" rock obviously depends on its mineralogy.

Much of the information published on wall rock alteration is in connection with hydrothermally deposited ore bodies (Grim, 1970). If hydrothermal alteration is intense (high temperature, long duration) the alteration product will tend to be the same regardless of the parent rock except for carbonate and quartzitic rocks. Limestones tend toward silicification, and dolomites tend toward the magnesium-rich clay minerals.

If hydrothermal alteration is slight, the primary minerals determine the end product. The magnesium-rich minerals (hornblende, biotite) change to chlorites. In the presence of alkalies, except potassium, the micas, the ferromagnesium minerals, and plagioclase tend to alter to montmorillonite. The presence of magnesium favors the formation of montmorillonite and potash favors micas. In general, basic igneous rocks tend to alter to montmorillonite. Kaolinite may form from any rock if the alkalies are absent or if the conditions are acidic and at a moderate temperature.

To summarize, it should be remembered that the alteration products are a function of the composition and pH of the invading fluids, their temperature and pressure, and of the composition of the material in the fault zone prior to the hydrothermal action.

Close to the surface, the filling material may have been exposed to chemical weathering. To a large extent, weathering and hydrothermal alteration will yield the same products. The difference is the depth of action of the two processes. Weathering occurs near the surface due to the influence of meteoric waters and air and hydrothermal alteration occurs at depth due to juvenile water. Weathering is therefore not a large factor in the formation of gouge.

Many factors control weathering, and it is difficult, if not impossible, to generalize the relative importance of the various factors. Weathering is the process of ion exchange whereby rocks and soils which are unstable in their enviornment are altered to a more stable product. The products of weathering are generally quartz, hydrated sesquioxides of iron and aluminum, and the clay minerals. The minerals of engineering importance are the clays, and their production depends primarily on the weathering enviornment and time.

Parent rock type is of course important, but mainly in the initial stages of weathering. Fillings containing kaolinite and fillings containing montmorillonite can both develop from the same parent rock given different climatic conditions and time. Conversely, the same clay can be developed from different rocks given enough time.

Kaolinite, the most common clay mineral, is formed under conditions which remove the alkalies and can form from any rock type. Kaolinite is the characteristic clay mineral in humid regions and is generally associated with acidic rocks.

Montmorillonite clays are formed in dry, cool climates under poor oxidizing conditions. Alkaline solutions favor their formation. The addition of potassium favors the formation of illite.

Perhaps the most important result of chemical weathering in discontinuities close to the surface is the leaching away of material, leaving "empty" discontinuities. Coupled with low confining stresses, such discontinuities will drastically reduce rock mass stability.

Finally, filling material may have been exposed to low-grade metamorphism during faulting. The ensuing products may later have been altered through hydrothermal action or weathering.

Sometimes clay fillings with a very low degree of consolidation have encountered at considerable depth (Brekke and Selmer-Olsen, 1965). Such occurrences seem to be hydrothermal products deposited in "pockets" in the fault zone. On the other hand, the well-arranged particles of clay minerals ensuing from hydrothermal alteration of e.g. feldspars may have an extremely high density.

Fault gouge is normally impervious, with the major exception being sandlike gouge. Otherwise, high permeability is usually associated with the rock mass immediately adjacent to the fault zone. This high permeability is due to the high degree of jointing often found adjacent to faults. As will be elucidated later, high water inflows in underground openings encountered when penetrating the impervious gouge is one of the most adverse conditions associated with faults.

V. STABILITY PROBLEMS CAUSED BY FAULTS AND SEAMS

A. Factors Influencing the Stability

The stability problems that faults and seams may cause underground are dependent on several factors. All of these interplay in the final behavior of the rock mass. It is therefore somewhat dangerous to isolate the different factors without viewing their influence in an overall context. However, it may be helpful to list some of the observations that can be drawn from documented experience regarding the influence of the different factors.

The first group of factors that can be listed relates to the opening itself, the size and shape, the method of excavation, and the method of support or reinforcement.

The influence of the size of the opening has two facets, namely on the standup time, and on the ultimate loads that might develop while the standup time problem is strictly a construction problem, the ultimate load that may develop can influence stability during construction as well as during operation.

Terzaghi (1946) found that the loads will increase linearily with size except for swelling grounds where high swelling pressures may develop regardless of the size of the opening. Either through the use of Terzaghi's assessment of rock loads, or more recently through the adoption of the so-called Kastner formula (Kastner, 1962), the effect of size has been considered in design of the support and lining systems as far as the ultimate loading is concerned.

From a review of the literature, it seems quite clear that the effect of the size on the standup time of gouge material is much more significant. The problems encountered increase drastically as the standup time increase exponentially with the active span (Lauffer, 1958). The active span is as mentioned the width of the tunnel or the distance from the supported part to the face if this is less than the width of the tunnel. While it may be feasible to drive a pilot bore through gouge material without problems that cannot reasonably be handled, it may be completely unfeasible to drive a large tunnel full face or even with a top heading through the same material without danger of collapse at the face. The case history from Straight Creek Tunnel clearly shows this. It does not seem possible at present to state in numerical terms when the size in a given zone simply is too large to handle and a multiple drift method of excavation becomes necessary. Such a decision rests fully on an engineering judgement in the individual case.

The method of support or reinforcement influence the outcome by how quickly they can and are applied after excavation, and the amount of deformation that they will allow. Using steel supports, a major problem has sometimes been to get invert struts in place close to the face.

The orientation of the opening in relation to the strike and dip of an individual fault or seam can have a considerable influence on the stability of the opening. In general, the problems increase as the strike becomes more parallel to the opening. However, even if the strike is across the opening, low-dipping seams and faults can represent a hazard. If the opening is driven from the hanging wall side, the discontinuity will first appear at the invert and it is most often possible to prepare for adequate support or reinforcement when driving through the rest of the zone. If, on the other hand, the opening is driven from the foot wall side, the zone will first appear in the crown, with the possibility of the wedge aligned by the zone and the tunnel falling in without warning.

Seemingly, the problems associated with a fault or seam would increase with the width of the discontinuity. However, this factor should always be assessed in relation to the attitude of the discontinuity and to the frequency, orientation, and character of adjacent joint sets, the existence of adjacent seams or faults (if any), and the competence of the wall rock type. As will be summarized later, several severe slides in tunnels have occurred where each individual seam or fault have been of a small width, but where the interplay between several seams and joints has led to instability.

On the other hand, it is rather obvious that as the width of a fault zone increases to the same order of magnitude as the size of the opening and beyond, the problem of excavating safely through the zone increases. At some point, the gouge material becomes the rather dominant factor over the competence of the wall rock and the properties of the discontinuities in the wall rock. It is no longer possible to "span over" the fault.

The water regime can influence the stability in several ways. It is generally accepted that driving through gouge with seeping water means heavier ground than gouge without seeping water. In running ground, water may lead to collapse of the face almost immediately upon excavation. In swelling ground, water may lead to free swell with subsequent loss of strength and wash-in or fallout, or to rapid mobilization of swelling pressures where the swelling clay is confined. Seeping water may dissolve porous of flaky calcite, and may thereby lead to instability.

It has been noted in several of the case histories that are summarized in the following chapter that influence of the water in terms of swelling, dissolving, and outwash is not necessarily obvious during construction. Therefore, a long time effect may easily be underestimated during the construction period, and the measures taken to insure long time stability prove inadequate.

As mentioned earlier, a real hazard exists where large quantitites of water in a permeable rock mass is released when an impervious fault gouge is punctured through excavation. In this instance, large quantities of gouge and rock can be released when the water inflow occurs.

The state-of-stress in a rock mass will as mentioned before not necessarily reflect itself in the density of gouge material. This seems particularly true when studying gouge on a "local" scale. On a larger scale, and in particular for very wide fault zones, it must be expected that the in situ stresses in the gouge increase with depth and also will reflect tectonic or remanent stresses in the area. Therefore, the loads that may develop through squeezing must be expected to be related to the overall in-situ stress for wide fault zones.

Finally, the stability of the opening will depend on the character of the gouge material.

B. Examples of Slides

Some examples of slides in tunnels associated with faults and seams are summarized below. The purpose is to elucidate the problems discontinuities may cause, both at the face and later.

Seelmeier (1959) reports on a slide that took place at the face of a 11.4 m^2 tunnel near Hieflauf in Austria. The general rock types encountered were dolomite, slate and limestone. An almost vertical "chimney" with a diameter of approximately one meter formed along the intersection of two faults to a height of 8 meters above the tunnel invert. The gouge material was brecciated limestone in a clay matrix.

Bistritschan (1956) reports on several slides that occurred in the pressure tunnel and diversion tunnel of the Sariyar Hydroelectric Plant in Turkey. The tunnels are situated in Paleozoic sedimentary rocks with some volcanics. Collapses took place in both tunnels due to a fault zone carrying heavily sheared serpentine, partly ground to clayey gouge. At the same place where a 200 m^3 slide occurred while the top heading was driven in the pressure tunnel, a 1500 m^3 new slide occurred during benching.

Brekke and Selmer-Olsen (1965) describes a number of slides in tunnels in Norway associated with the presence of swelling clay. One slide developed at Tunnsiødalen during the excavation of a 35 m^2 tailrace tunnel through a system of thin fiederspalten carrying montmorillonitic gouge. In addition, the feldspars in the wallrock had been altered to montmorillonitic clay. During excavation, the fiederspalten seemingly did not represent a problem. In an area with seeping water, however, swelling with ensuing sloughing and sliding started within one week.

The recognition of potential problems of this type is often difficult when the gouge and wallrock is dry and appear to have considerable competence.

Wahlstrom (1948) mentions several examples attesting to this, as do Rossar (1965) and Morfeldt (1965).

At the Tokke IV powerplant, a montmorillonitic breccia immediately flowed into a 35 m^2 tunnel after blasting; 150-200 m^3 of this material filled up the face of the tunnel. The tunnel was then driven around the sliding area at a distance of approximately 25 m. When the same occurrence was encountered again, the montmorillonitic material had a considerably higher consolidation. This is an example of the variation in consolidation that can be found locally within the same occurrences.

A 4 km long water tunnel (5 m^2) in Caledonian rocks at Skogn, north of Trondheim, was driven through a zone of montmorillonite-infested rock. After one week, sliding occurred, and the tunnel was almost completely filled up with slide masses over a distance of 40 meters.

A slide involving partially collapse of an unreinforced concrete lining, occurred in the Kvineshei railroad tunnel in 1948, 8 years after the tunnel had been finished. A pipe-like cavity with a diameter of 4-6 meters and a height of 34 meters above the railroad tracks had developed along the intersection of a montmorillonite - carrying joint and a brecciated zone of porous calcite and partially altered granitic blocks. It was assessed that the porous calcite through the years had dissolved, giving space for free swell of the montmorillonitic material. This again lubricated the mass in the "pipe." An ensuing earth pressure on the lining, possibly combined with hydrostatic pressure, led to the partial collapse of the lining.

In a headrace tunnel at the Hemsil I powerplant, driven through Caledonian rocks, a dominant system of faults and seams with a direction approximately perpendicular to the tunnel axis was encountered. The thickness of the individual seams and faults varied from under 1 cm to zones up to 8 meters. During the construction period, the presence of montmorillonite in the filling material was realized, and the parts of the tunnel that were considered dangerous were lined with concrete. Smaller and more separate seams were sealed with unreinforced shotcrete. After the tunnel had been in operation for about 8 months, a slide of about 200 m^3 with up to 3 m^3 blocks took place in 1960. A 2-1/2 meter wide section along the tunnel that was inter-woven with thin, chloritic joints between two montmorillonite-carrying seams, each a few centimeters thick, had been released along these seams and had slid out and filled the tunnel. The slide went approximately 9 meters up from the tunnel roof, corresponding with the hydrostatic pressure at this place when the tunnel was filled with water during operation. At several other places in the tunnel, "puncturing" of the shotcrete had also taken place but without causing more serious slides.

Detzlhofer (1968) discusses slides in several pressure tunnels. In the 12 m^2 Kauner Valley pressure tunnel in Austria, 5 slides occurred upon de-pressurizing a 1.4 km long test section that was partly reinforced by shotcrete and wire mesh. The crystalline rock mass was intersected by numerous seams and joints, many with clay fillings and slickensides. The sliding masses for the different slides ranged from 90 m^3 to 1550 m^3.

Detzlhofer also discusses the failures found after 7 years of operation in the Kemano-Kitimat pressure tunnel in British Columbia, originally reported by Cooke et. al. (1962). The major failure involved 19,000 m^3 of slide masses. Faults as well as many minor discontinuities were filled with easily erodable gouge and partially with soluble calcite. Shotcrete as applied proved inadequate in ensuring long-time stability.

One of the most severe cases of high water inflow associated with faults occurred in the San Jacinto tunnel near Batting, California (Thompson, 1966).

A number of faults were intersected by the tunnel. The hanging wall side of the faults was heavily jointed and highly permeable. Driving the tunnel towards the hanging sides of the faults did not involve very serious problems since the rock mass was drained over a period of time prior to reaching the fault. However, when the faults were approached from the foot wall side, no pre-drainage could take place due to the impervious gouge material in the faults, and sudden inflows of water occurred. The maximum flow from one point reached 16,000 gpm. and from all headings the peak flow was approximately 40,000 gpm. Up to 3,000 cubic yards of crushed material surged in together with water through the ruptured diaphragm represented by fault gouge. Water pressures of 600 psi were measured in a few instances, and pressures ranging from 150 to 300 psi were common.

VI. TESTING PROGRAM AND CASES STUDIED.

A. Testing Program

The purpose of the testing program is to study the properties in detail of selected gouge materials in order to establish some simple methods and techniques that may yield pertinent information on the potential behavior of the gouge material. It is appreciated that large-scale in situ tests, and carefully executed triaxial tests on undisturbed samples may yield more accurate information then can be obtained by the simple tests being evaluated. The advantage of the latter is that they can more easily be done and therefore find wider application.

Sampling

Sampling is carried out using a polyutherane foam for protection both against mechanical disturbance and against loss of porewater. The samples are wrapped in aluminum foil and polyethylene sheets, placed on polyutherane stock in cardboard boxes, and then foamed. This method has worked very well. One advantage is that test specimens for direct shear testing can be cored through the foam, which acts as a confinement and prevents the block to be cored from falling apart.

Block samples were taken immediately after excavation whenever possible. These samples are quite close to being undisturbed, although the effect of blasting and the immediate relaxation towards the opening may have led to some remolding.

Composition

X-ray diffraction is being used as the major method for mineral identification, using total sample crushing and sedimented specimens (approximately 20 mg per square inch).

Additionally, DTA and dyes tests are available for mineral identification.

Gradation

Standard techniques are used to find the grain size distribution for the samples. Of major interest is the clay content in the samples.

Atterberg Limits

The liquid limit, plastic limit, and shrinkage limit are the water content between respectively the liquid/plastic, plastic/semisolid and semisolid/solid state. The testing rocedure is standardized.

It has long been recognized that the Atterberg Limits of a argillaceous soil correlate quite well with potential behavior of the soil. In this regard, Terzaghi (1926) wrote "If we know the three limits of a soil we are already in a position to compare this soil with others and can at least anticipate what its properties may be. If we know in addition the results of physical tests performed on another soil with fairly identical limits we can say the soil is known."

Skempton (1953) defined the ratio of the plasticity index to the clay fraction content as the "activity" of the clay. He showed that activity is related to mineralogy and geologic history of clays and to the portion of the shear strength called true cohesion.

It is hoped that the Atterberg Limits and the activity of clayey gouge materials will prove to be significant parameters for assessing their potential behavior.

Direct Shear Test

Obtaining reliable values for the strength of a gouge material is difficult because it is very hard to retrieve undisturbed samples. .The in situ state-of-stress in the gouge is usually unknown and the correct confining pressure for testing, is therefore difficult to assess, and finally it is questionable how representative the test specimens are for a given gouge in view of the nonhomogeneous nature of such materials.

The direct shear test was selected as the appropriate test to obtain strength data for the sampled fault gouge. Valuable additional information can also be obtained from this test. Samples can be consolidated in the direct shear apparatus to obtain the compressibility of the material. The maximum past pressure that the gouge has been subjected to can be approximated from the consolidation curve. Such data might approximate the in situ stresses normal to the fault.

Testing is carried out in a Carrol-Warner direct shear apparatus at a strain rate of 0.2"/minute. Both the initial and residual strengths of the gouge are measured.

Swelling Tests

The relative potential swellability is being measured through a free swell test and a swelling pressure test, using a Geneor swelling apparatus (Brekke, 1965). The tests are performed on specimens of the clay fraction in the gouge.

Slaking Test

Recently, a slaking apparatus developed at Imperial College in London has been obtained. This apparatus is basically designed to test the durability of shales. However, it is felt that it may be well suited for testing the slaking characteristics of gouge as well.

Penetration Testing

Penetration tests are fairly rapid and simple to perform and may prove to be a valuable tool in assessing the physical properties of gouge.

For a purely cohesive material ($\phi = 0$), the bearing capacity is defined by the equation:

$$q = cNc + P \qquad \text{(Meyerhof, 1951)}$$

where q = the ultimate bearing capacity, or penetration resistance

c = the undrained shear strength

Nc = the bearing capacity factor

P = the overburden pressure

Generally Nc is taken as 9.5 and the overburden pressure, P, is neglected, and the equation becomes

$$c = q/Nc = q/9.5$$

However, most soils are not purely cohesive and the value of Nc does not remain constant.

In recent studies of the penetration resistance of soils a method has been developed by Durgunoglu (1972) which determines the ϕ and c strength relationship from penetrometer data. In order to obtain specific values of ϕ and c, however, the method requires that one of the two be defined by other means.

A field penetration device was developed for field testing. The device is hydraulic and consists of a small pump, a pressure gage, and a piston. Several lengths of threaded pipe are used to position the piston prior to penetration. Pictures of the device were included in the semi-annual report.

During field use in the Straight Creek Tunnel the penetrometer proved to be slightly smaller than that needed to obtain good results. The piston rod deflected excessively during penetration. However, the results obtained are useable. A stronger device will be used in future field work.

Apart from that already mentioned, interpretation of the stress-penetration curve can take several forms. Houston and Namiq (1970) correlated the slope, G, of the curve with the density of a basaltic lunar soil simulant. Another correlation might be the area under the curve.

B. Cases Studied

Straight Creek Tunnel

The Straight Creek Tunnel (Colorado Division of Highways) was selected for an extensive study and sampling program. The tunnel site is in central Colorado about 55 miles west of Denver. The tunnel will carry Interstate 70 traffic under the Continental Divide. The tunnel presently under construction is the future west bound lanes. A second tunnel for the east bound lanes will be constructed later.

Robinson and Lee (1962, 1963, 1965) did the major geologic effort prior to and during construction of the pilot bore; a 13 foot diamter bore used for exploration purposes. Abel (1967) used data from the exploration tunnel for a statistical evaluation of tunneling conditions. Scott et. al. (1968), correlated data from seismic refraction and electrical resistivity measurements with tunneling conditions. A wealth of additional information is available in unpublished reports from the Colorado Division of Highways and the U.S. Bureau of Reclamation.

The rock of the area consists of Precambrian granite and a variety of biotite gneisses which occur as inclusions in the granite. The bedrock has been extensively faulted and sheared and locally altered. The bedrock is overlain by thin deposits of alluvium, talus and landslide materials of Quaternary age.

The granite rock is light gray with, in some cases, a pinkish tint. It is composed of plagioclace, microcline, quartz and biotite. Irregularly shaped pegmatite bodies occur within the granite and consist of pink microcline and quartz with some biotite or muscovite.

The metasedimentary rocks are fine to medium grained and with plagioclase, quartz and biotite as major constituents. The rock is foliated with

with layers up to 1 inch in width. Locally, the biotite is altered to chlorite, and in shear zones the feldspar is altered to clay.

The extensively sheared and altered zones are associated with the Loveland Pass Fault that intersects the tunnel at approximately the mid-section. The zone consists of numerous faults or shear zones with varying character. Generally, fault gouge in granite environment appears as a coarse sand with typical running or flowing character. In some cases, the feldspars have been altered to clay. In the metamorphic rocks, the gouge is generally chloritic clay.

A number of different instruments have been installed in the tunnel with the objective of monitoring the behavior of the rock and the support systems. The data obtained will be used to correlate laboratory-measured properties of the material, in-situ measured data, and field observation of problems during construction.

Gloetzl cells have been installed between the rock and the concrete liner in order to measure the loads developed as construction proceeds and later. Microdot strain gages have been welded to the steel rib supports and reinforcing bars in order to provide information on the development of strains in the primary supports and later in the final support system. A fairly simple type of instrument is the tape extensometer, used to obtain measurements of rock or support movement. The deformation measurements are then plotted against time to give information on the time rate of movement and if and when movement stops.

Sampling was carried out in several of the multiple drifts as they advanced to insure that there would be as little moisture loss as possible. Block samples were carefully cut from the walls, sealed in aluminum foil and plastic bags and further protected by polyutherane foam before they were removed from the tunnel. The samples are presently being tested in the laboratory. Test results which are available at this time are presented in Appendix A.

The results of the field penetration tests are included in Plates 1A through 5A. Test 1 and 2, Plate 1A, were taken in a soft chloritic clay gouge in an area with typical squeezing behavior. Tests 5, 6, 7, 8 were performed in a green clayey sand-like gouge in a 2 foot wide shear zone through granite rocks. This material was wet and showed typical running behavior. Tests 9 and 11 on Plate 3A represent the results of tests in altered granitic rocks. The material is a clayey coarse sand giving moderate pressure on the sets and with a ravelling-type behavior. Test 12 was taken in a small green clay seam between shist blocks and tests 13 and 14 were in altered granitic material.

Plates 6A and 7A give the results of grading analysis for Samples No. 14 and 15. Both samples were taken at the face of the south wall plate drift at Station 83+25. Further testing is underway on this material.

Twin Buttes Mine, Arizona

The Anaconda Company has provided much data associated with a system of faults through a large open pit mine in southern Arizona. The problem at the site is caused by the intersection of two families of faults and the southern face of the pit. Small wedge slides occurred along this face; the slides taking place along the fault planes, and have cumulated in the instability of the entire southern wall. The Anaconda Company has undertaken an extensive investigation into this problem and have provided data and samples to this project. The data has been tabulated and is presented in Appendix B, Plate 1B.

The data has been analyzed in terms of correlating the strength-consolidation pressure with the Atterberg Limits and the percent material passing the 200 sieve size.

The assumption in this analysis is that the direct shear test approximates a consolidated-undrained condition. With this assumption, the strength and consolidation pressure can be obtained. The assumption was not valid in the case of the Twin Buttes Mine samples because they were desiccated prior to, and not saturated at the time of testing. The correlation obtained were poor. However, other correlations will be attempted since it is felt that the testing performed represented the in situ condition at the time of failure.

The mineralogy of the samples were obtained by X-ray defraction techniques. The results of this study are given in Appendix B, Plate 2B.

Strength test data for the Twin Buttes material is given on Plates 8A and 9A and summarized on Plate 10A. Plate 11A presents the results of a consolidation test performed on this material. Plate 12A is the grading analysis. The material is a mottled tan and grey silty clay with small rock fragments.

Auburn Dam Site, California

The Auburn Dam will be a thin, double curvature concrete arch dam about 685 feet high with a crest length of about 4000 feet.

The site of the dam is in a broad V-shaped canyon of the North Fork of the American River. This region is cut by several northwest striking faults. Wallace and others (1969) report that the foundation of the dam consists of a steeply inclined sequence of metamorphic rock made up mainly of amphibolite. Talcose and chloritic schists with some serpentine make up the remainder. Discontinuities include faults, shear zones, joint-set and cleavage planes.

The test program carried out by the Bureau of Reclamation included in situ jacking tests, both radial and uniaxial, and in situ shear tests on joints, shears and other planes of weakness. Laboratory tests were also performed on gouge and further testing is presently underway at Berkeley. Test results for the Auburn Dam Site are given in Appendix A Plates 13A through 20A.

Direct shear test results are given on Plates 13A through 17A. Plate 18A is the Shear Test Summary followed by the consolidation test results on Plate 19A and the grading analysis on Plate 20A.

Nast and Hunter Tunnels

The Nast and Hunter Tunnels area part of the Bureau of Reclamation's Fryingpan - Arkansas Project. This project is a multipurpose water development in central and southeastern Colorado. The purpose is to divert water from the western slope watersheds in the Colorado River Basin to the Arkansas River Basin east of the Rocky Mountains.

The Nast Tunnel is 3 miles long through a light grey quartz monzonite. The tunnel is advanced using a 9'9" diameter German made tunneling machine. The Hunter Tunnel is to be 7 miles long and is being advanced using conventional methods. In both tunnels these will be sections with no support, shotcrete, and concrete lining.

Both tunnels are through a relatively homogeneous granite-quartz monzonite. Gouge material is associated with generally small discontinuities although there are some shear zones up to 200 feet in width.

These zones contain a highly altered fault breccia that swell or squeeze into the tunnel at a rate of about 1 inch per month. The zones are steel supported.

Standup time is another problem associated with the gouge, being slightly over 2 hours at which time the material starts to spall and slough into the tunnel. This has been a problem in the Nast Tunnel because it requires more than 2 hours to advance the tunneling machine i full length.

These two tunnels afford an excellent means to compare tunneling methods as related to stability problems. Samples of gouge are presently being collected by Bureau personnel as faults and seams are encountered.

Additional Data

The U.S. Army Corps of Engineers encountered slope stability problems at the Libby Dam site. These problems are associated with sliding along clay filled bedding plane joints. The Corps of Engineers has recently kindly provided the data obtained from testing the gouge including gradation curves, strength tests and consolidation tests.

VII. A TENTATIVE CLASSIFICATION OF GOUGE MATERIAL

Based on the preceeding discussion, a tentative classification of gouge material is presented in Table 1. The basic division is made according to the material that dominates the properties of the gouge. It is not necessarily the most abundant material.

It is clearly understood that gouge in a given fault zone being a complex material may well overlap several of the classes suggested, e.g., porous or flaky calcite bands in a swelling clay matrix.

There is also often a rather subtle distinction to be made between "swelling clay" and "inactive clay." In some clayey gouges, a small amount of montmorillonitic material may dominate the behavior, and the gouge therefore be classified as swelling. In other instances, the clayey gouge may be rather inactive in spite of a substantial content of montmorillonitic material.

The third group is intended to cover blocky gouge material that is heavily interworn with slickensided seams and joints filled or coated with the minerals listed. The characteristic property of such gouge is low shear stren[illegible], in p[illegible]ular when wet.

The fourth group covers gouge with a typical cohesionless behavior. The major problem associated with such gouge is the short stand-up time.

The fifth group involves only long-time stability.

The potential behavior is listed in summary for "at face" and "later." It is common to distinguish between loads on the primary support, reinforcement, or lining system installed to ensure stability during construction, and loads in the final reinforcement or lining usually added to ensure stability for the lifetime of the structure. The time expired between the two stages can, however, vary considerably from casting of the final lining at the face to casting several months after excavation. It is therefore considered more meaningful to use the proposed distinction. "At face" relates to the difficulties involved in driving through the fault zone, "later" to the adequacy of applied methods of support, reinforcement and lining.

The third column in Table 1 lists the pertinent properties and parameters associated with the different types of dominant material. It has not been considered feasible at this stage to quantify the properties.

The last column includes all factors that influence the actual behavior in situ of the gouge material.

Table 1

Tentative Classification for Fault Gouge Material

Dominant Material in Gouge	Potential Behavior of Gouge Material: At Face	Potential Behavior of Gouge Material: Later	Pertinent Properties and Parameters
Swelling clay	Free swell, sloughing. Swelling pressure and squeeze on shield.	Swelling pressure and squeeze against support or lining, free swell with down-fall or wash-in if lining inadequate	Relative swellability Free swell and swelling pressure Composition Texture Density and water content Strength Deformability
Inactive clay	Slaking and sloughing caused by squeeze. Heavy squeeze under extreme conditions.	Squeeze on supports or lining where unprotected, slaking and sloughing due to environmental changes.	Relative slakeability Composition Texture Density and water content Strength Deformability
Chlorite, talc, graphite, serpentine	Ravelling	Heavy loads may develop due to low strength, in particular when wet.	Composition Texture Density and water content Strength Deformability
Crushed rock fragments or sandlike gouge	Ravelling or running; standup time may be extremely short.	Loosening loads on lining. Running and ravelling if unconfined.	Composition Gradation Density and water content Strength, in particular cohesive contribution, if any Permeability
Porous or flaky calcite, gypsum	Favorable condition	May dissolve, leading to instability of rock mass.	Permeability Solubility Texture

Factors Influencing Actual Behavior
The size and shape of the opening.
The method of excavation.
Method of support, reinforcement or lining.
The orientation of the opening in relation to the strike and dip of the fault or seam.
The width of the fault zone.
The existence of adjacent seams and faults (if any).
The frequency, orientation and character of adjacent joint sets
The competence of the wall rock type.
The Gouge Material
Time elapsed after excavation.
The in-situ state-of-stress.
The water regime.

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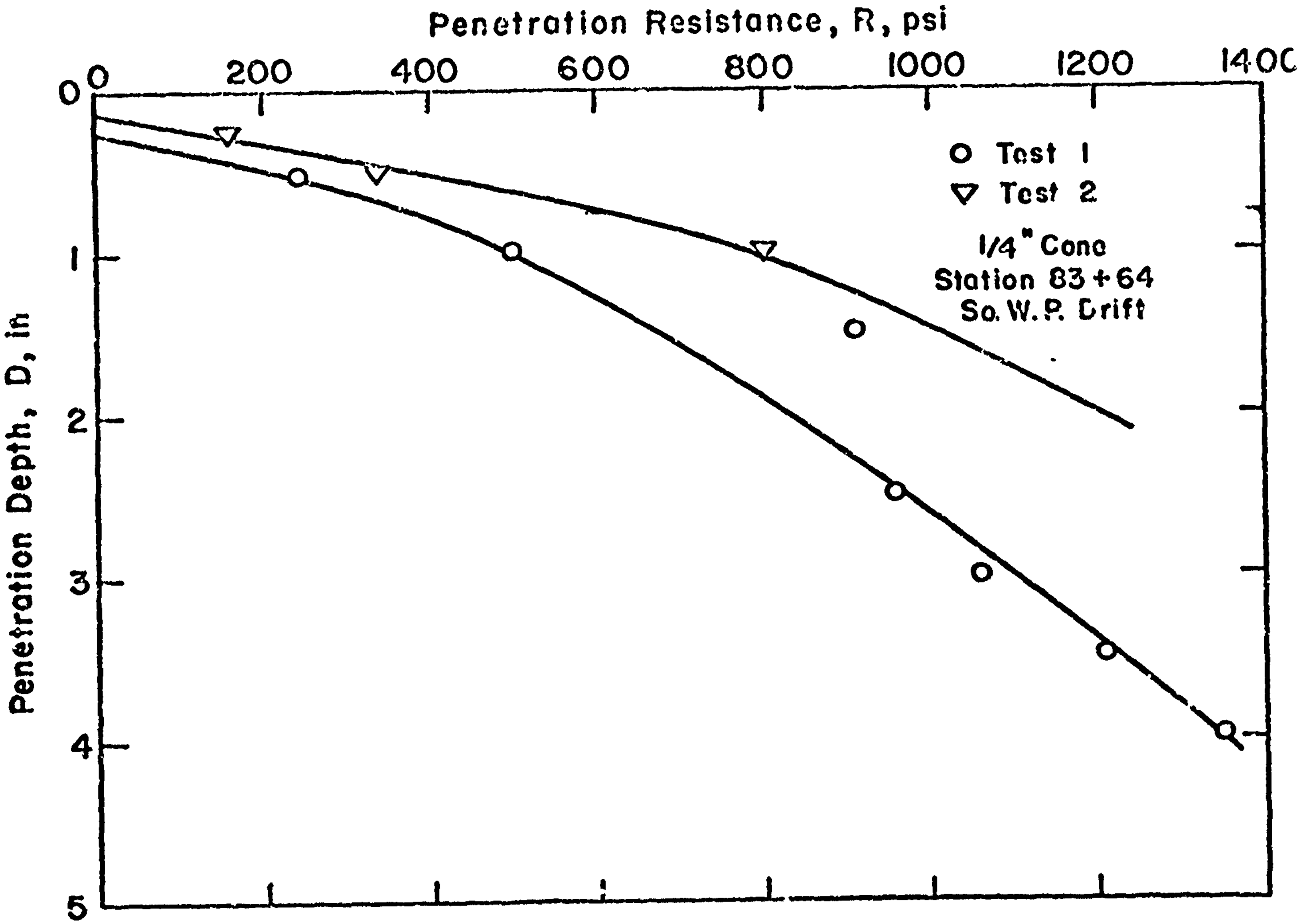
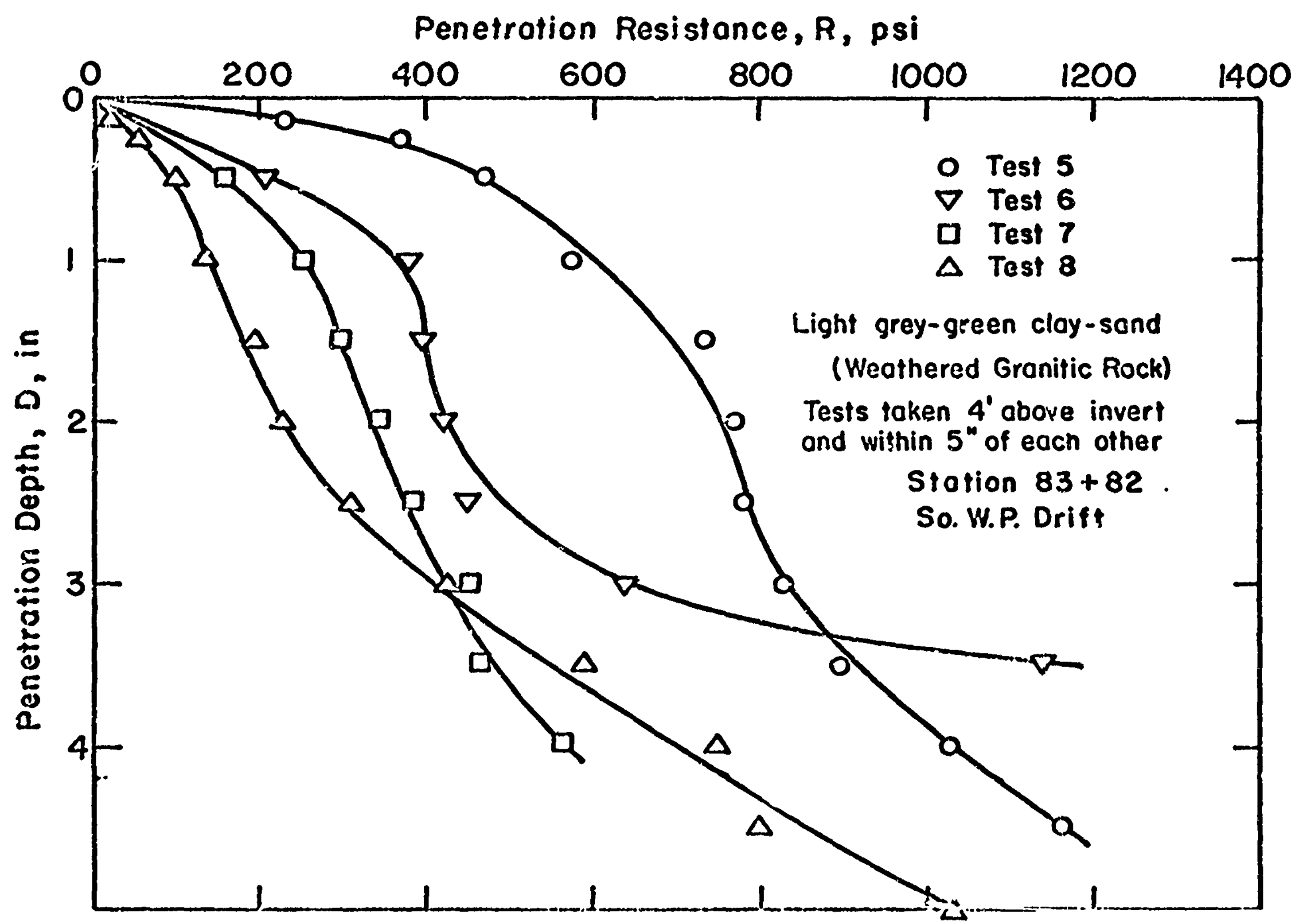
APPENDIX A

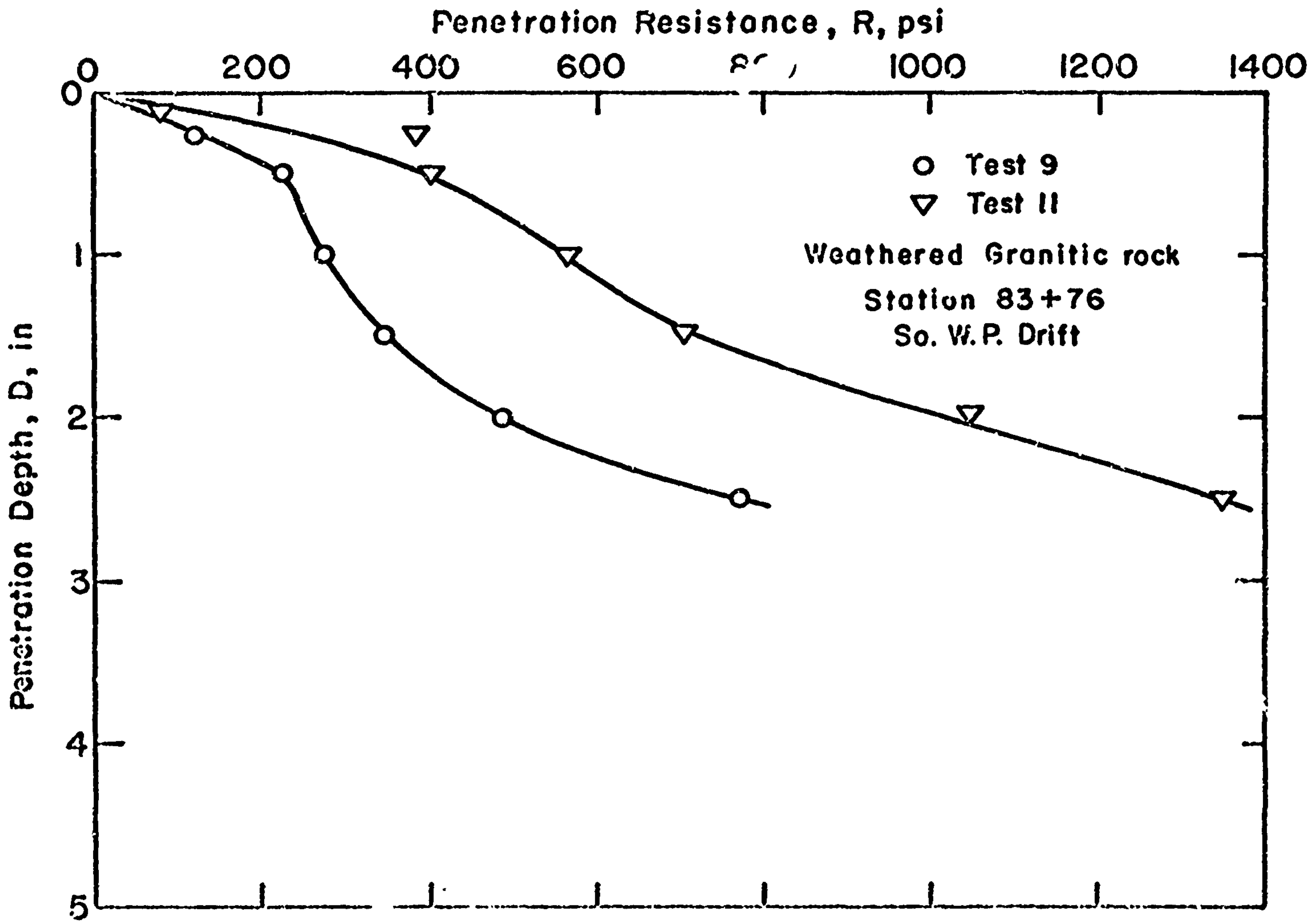


IN SITU PENETRATION TEST RESULTS - STRAIGHT CREEK TUNNEL



IN SITU PENETRATION TEST RESULTS - STRAIGHT CREEK TUNNEL

PLATE 2A



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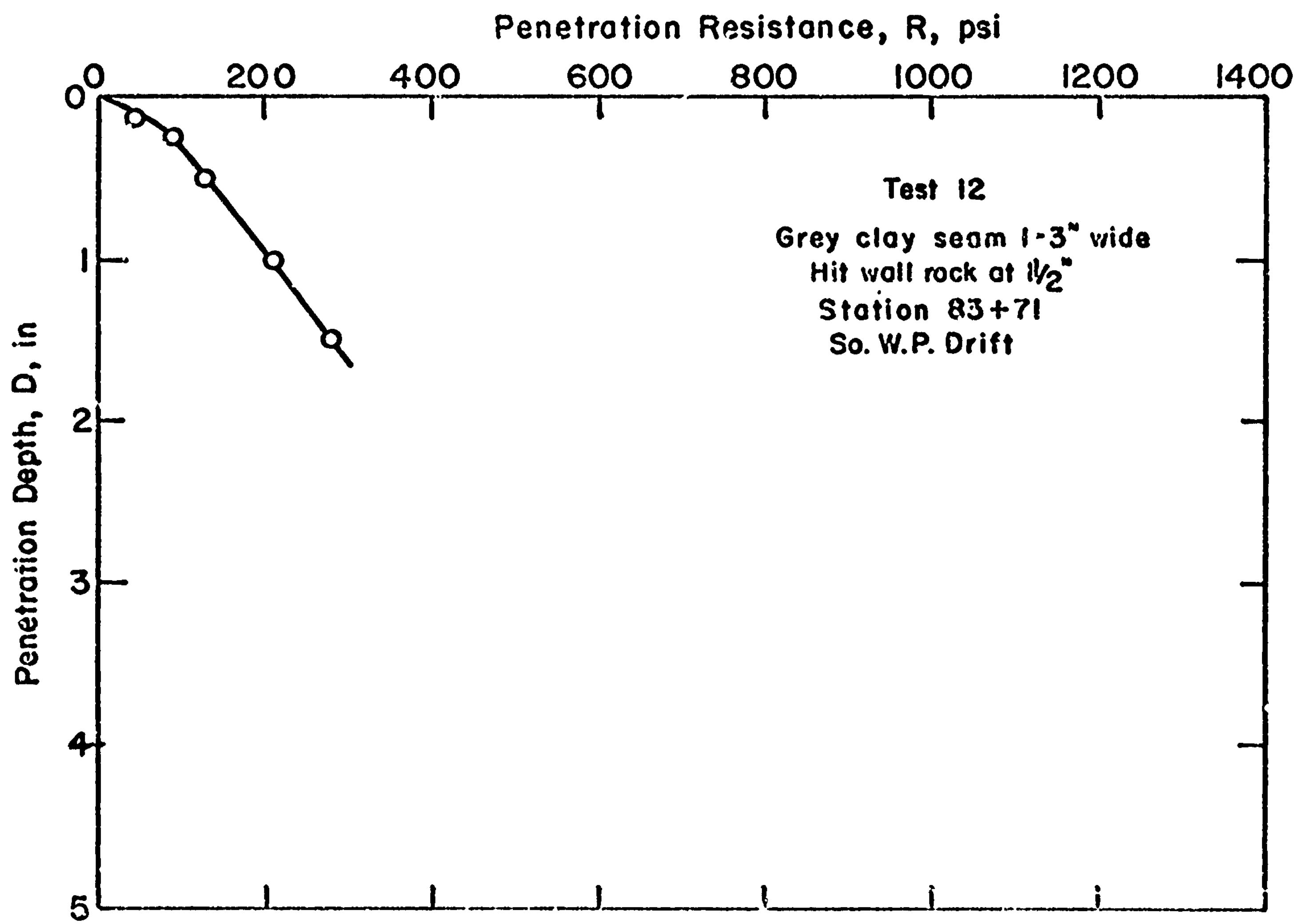

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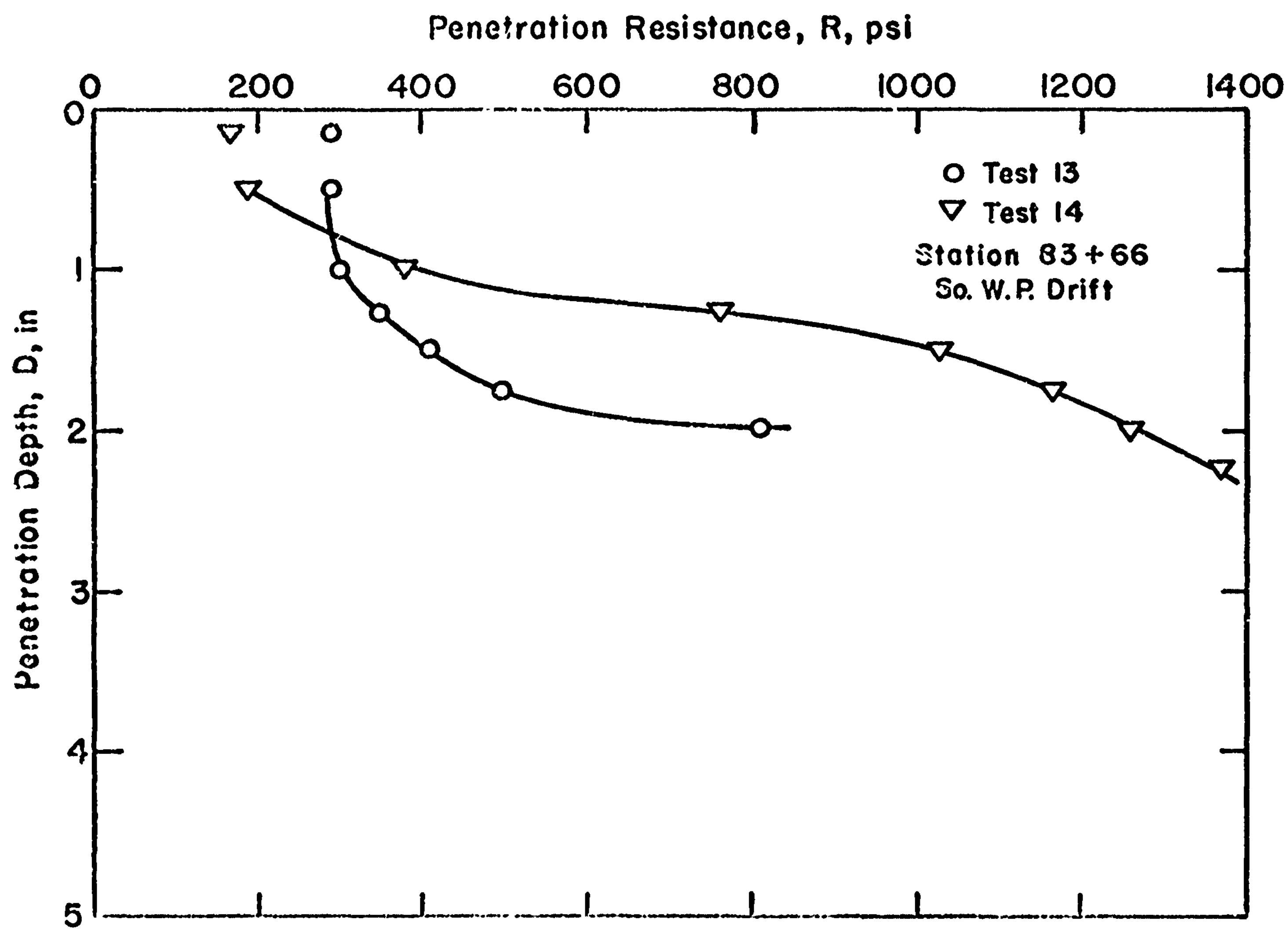


IN SITU PENETRATION TEST RESULTS - STRAIGHT CREEK TUNNEL

GRADING ANALYSIS

STRAIGHT CREEK TUNNEL
Sample No. 14

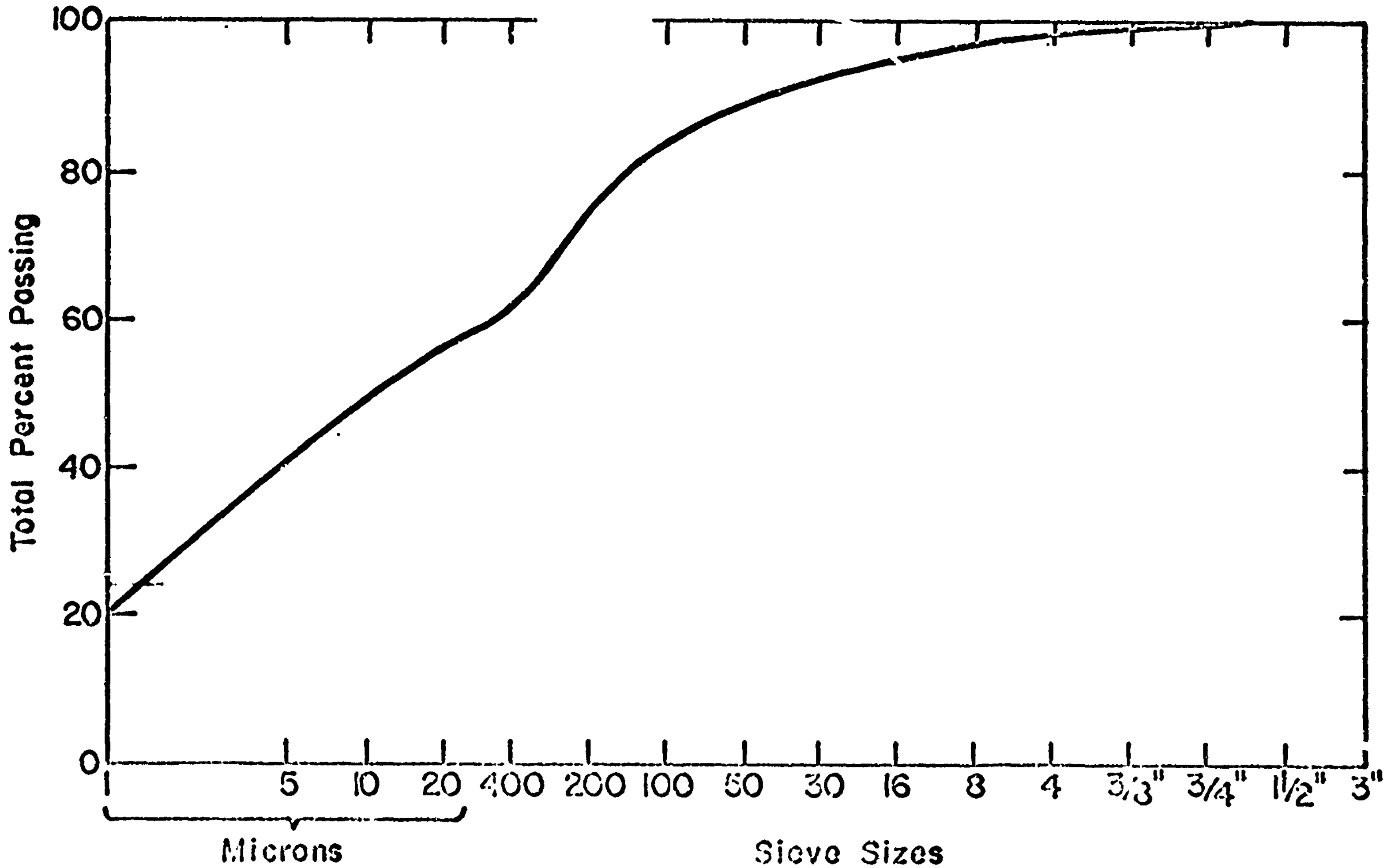


PLATE 6A

GRADING ANALYSIS

STRAIGHT CREEK TUNNEL
Sample No. 15

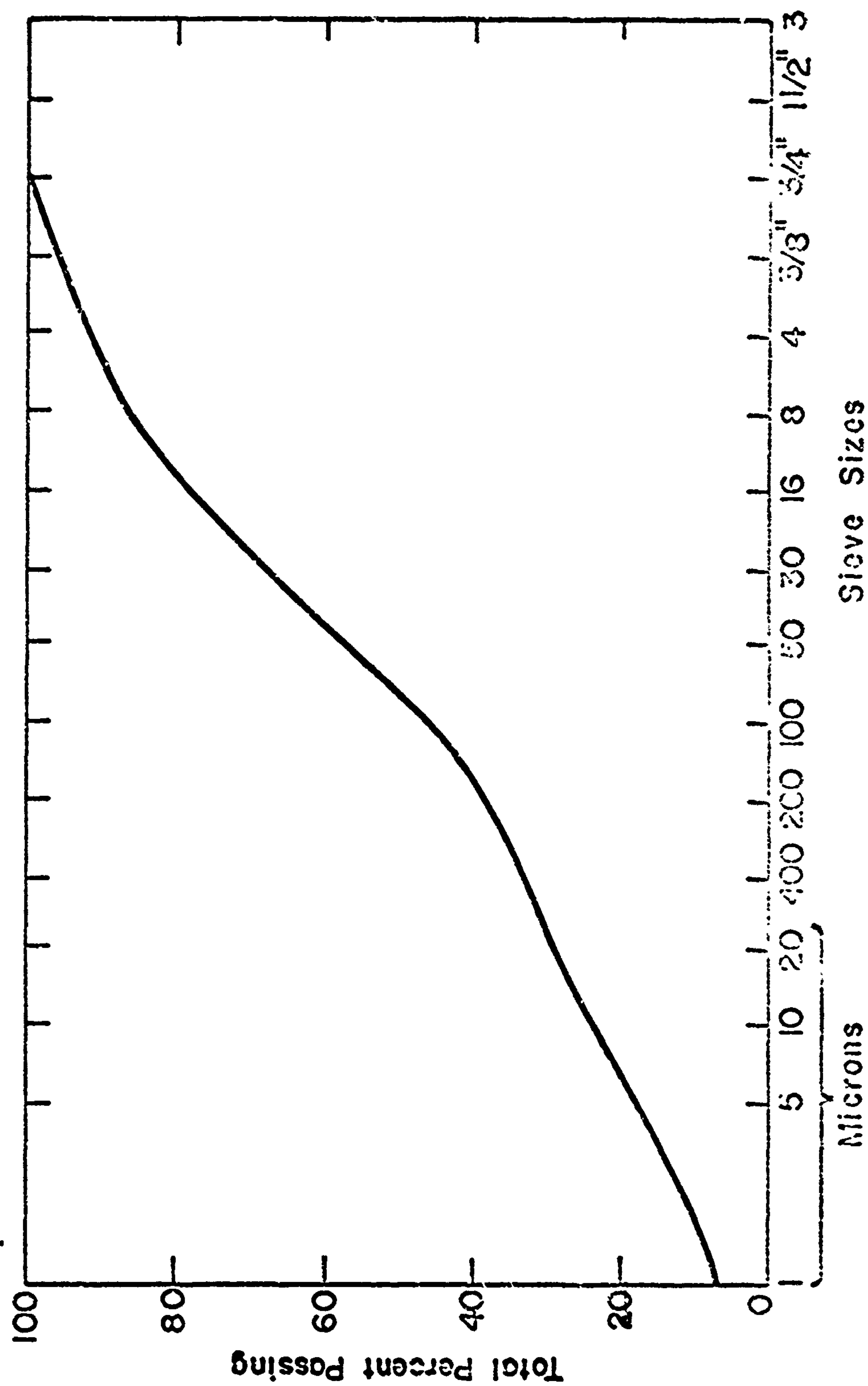


PLATE 7A

DIRECT SHEAR TEST RESULTS

TWIN BUTTES MINE
Sample No. 22L

Normal Stress	-	4 ksf
Moisture Content	-	32%
Dry Density	-	97 pcf
Void Ratio	-	.73

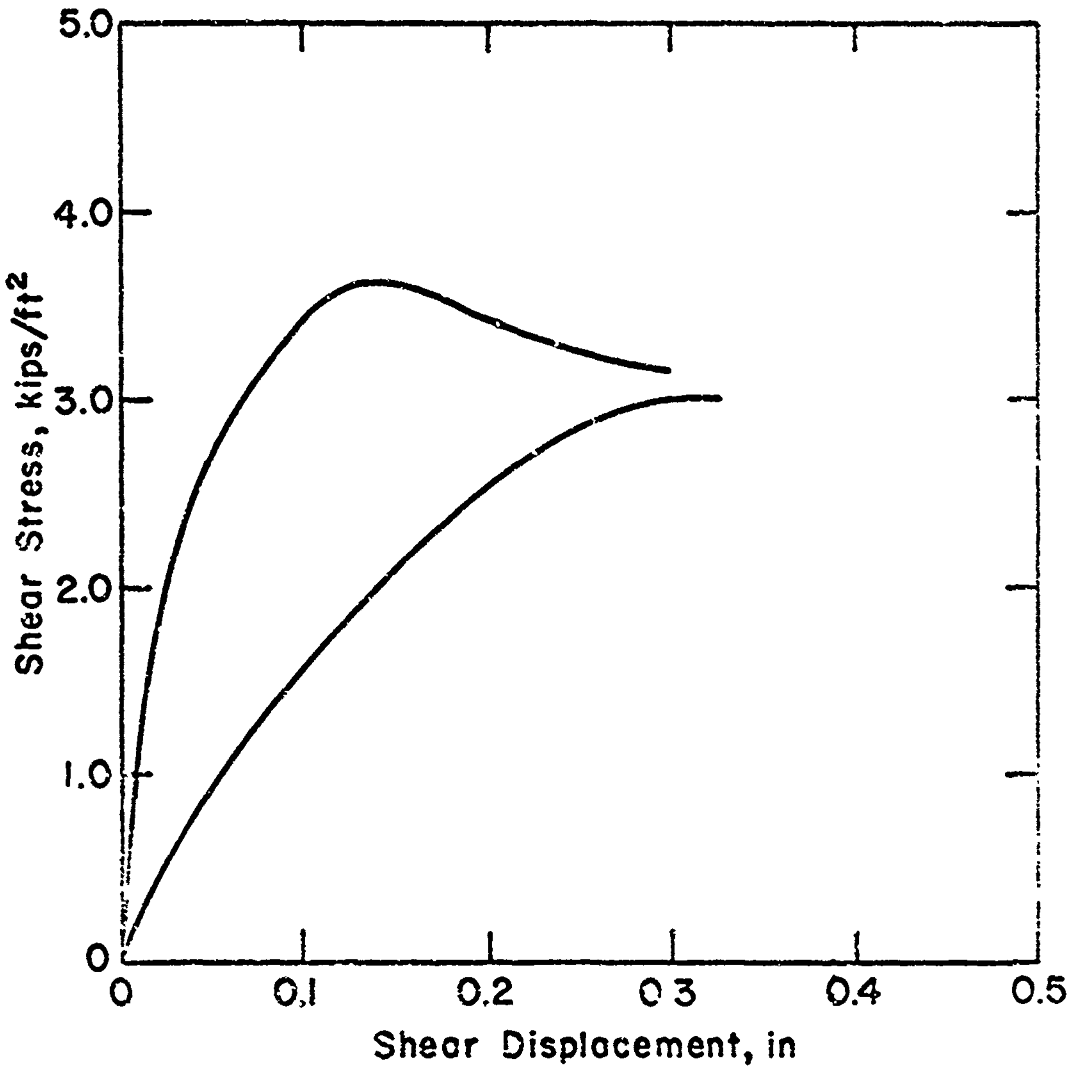


PLATE 8A

DIRECT SHEAR TEST RESULTS

TWIN BUTTES MINE
Sample No. 22L

Normal Stress	-	50 ksf
Moisture Content	-	21%
Dry Density	-	103 pcf
Void Ratio	-	.61

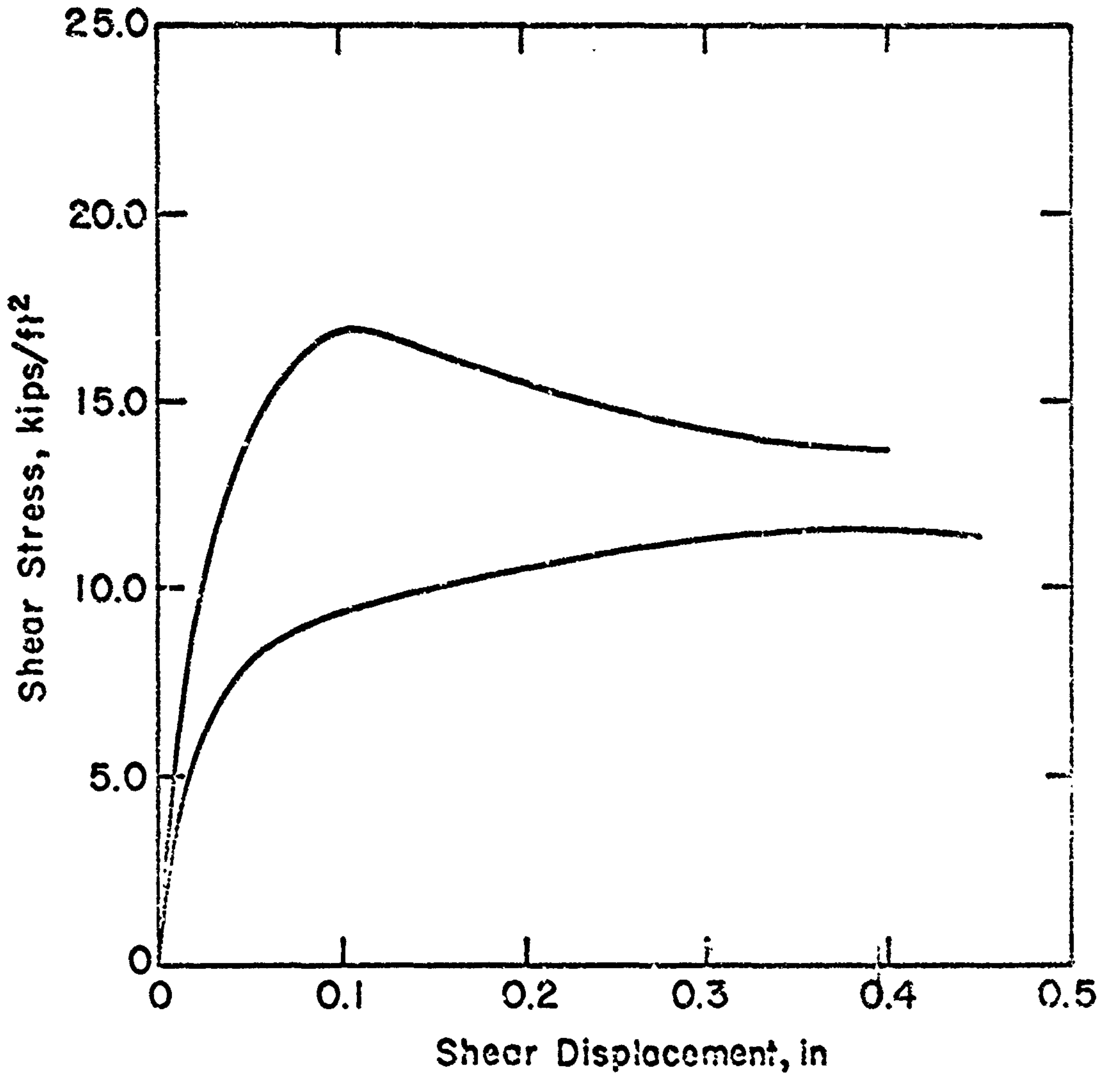


PLATE 9A

SHEAR TEST SUMMARY

TWIN BUTTES MINE
Sample No. 22L

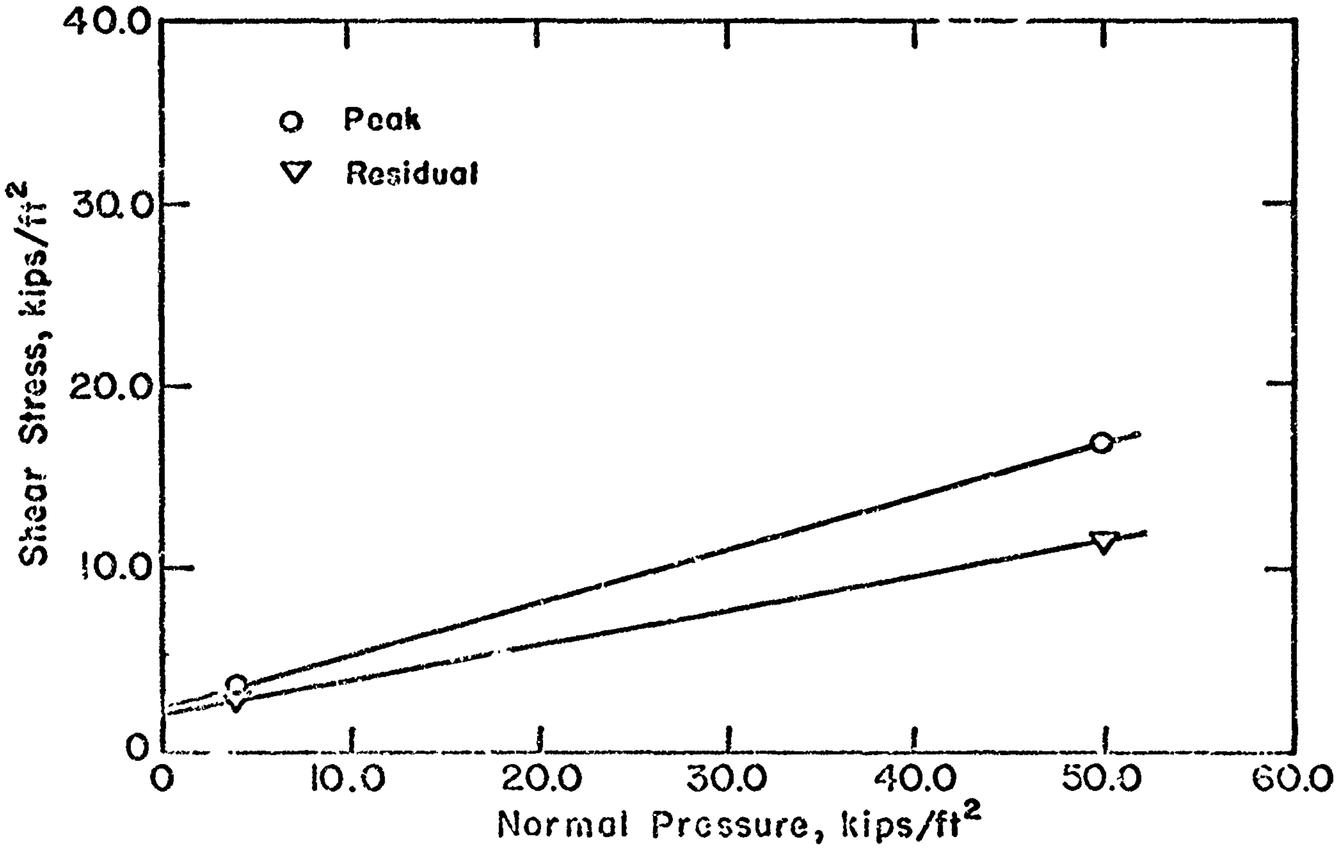
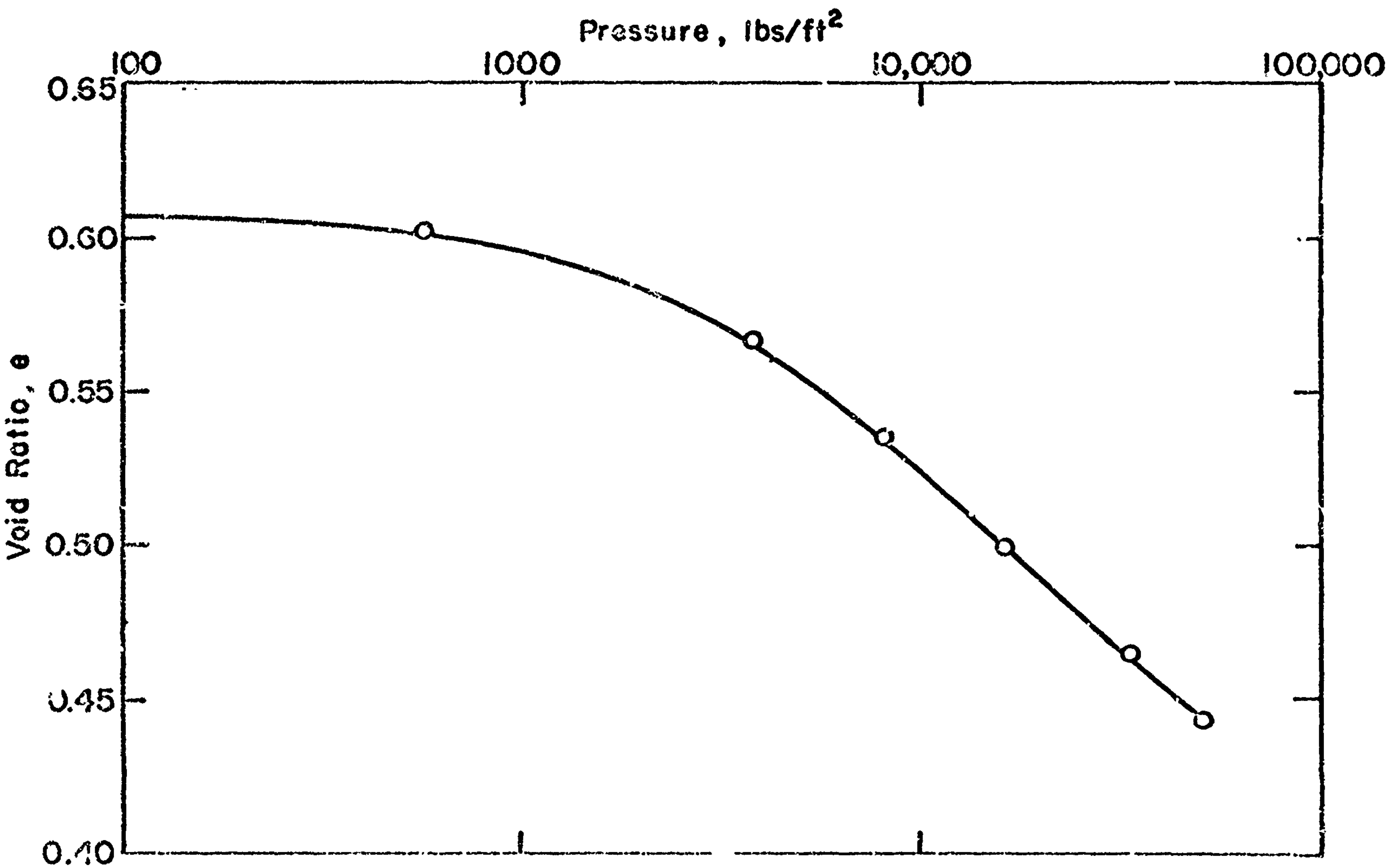


PLATE 10A

TWIN BUTTES MINE
Sample No. 22L

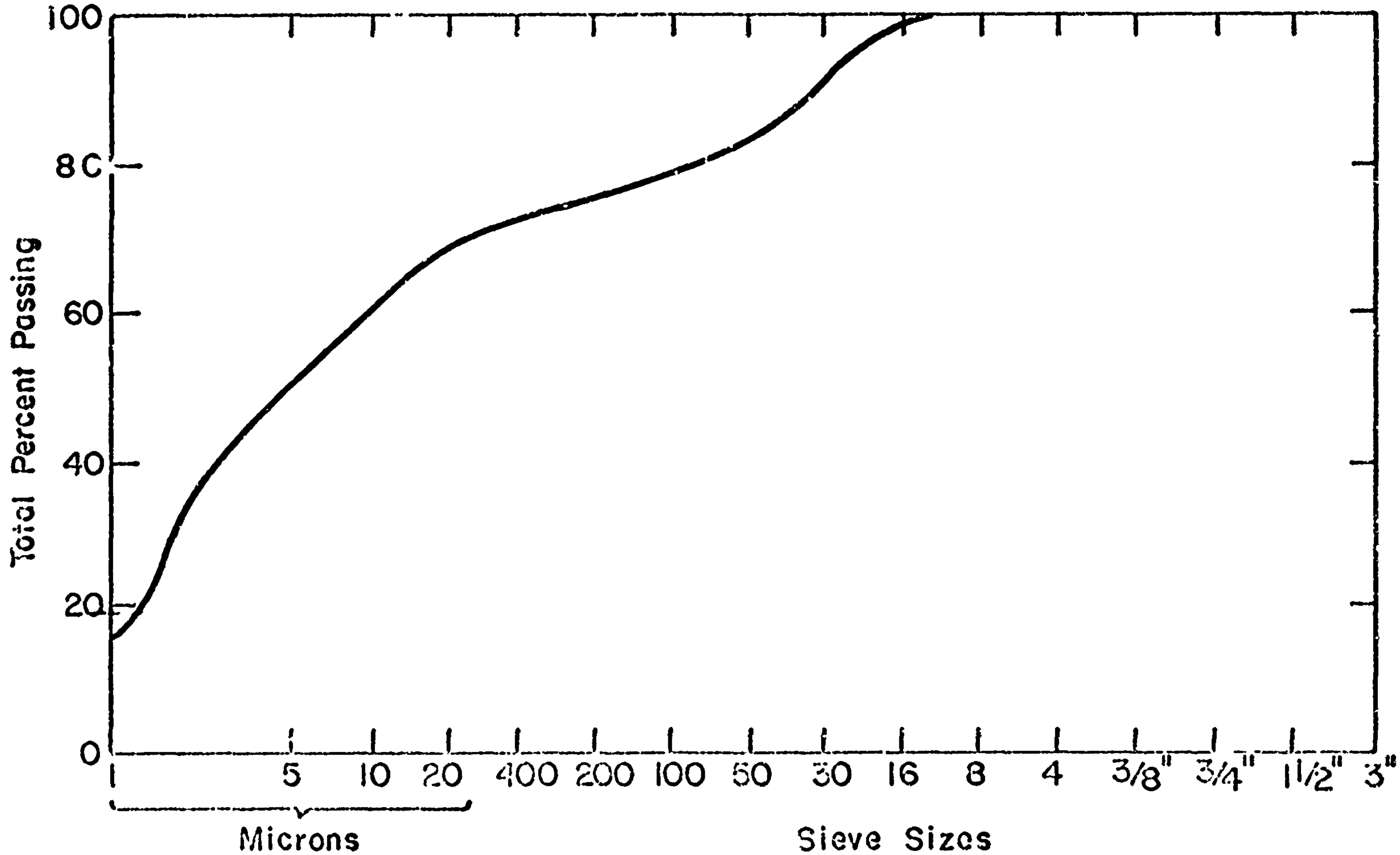
Atterberg Limits		
LL	-	45
PL	-	22
PI	-	23
A	-	.74

Specific Gravity	-	2.65
Void Ratio	-	0.61
Moisture Content	-	21%
Dry Density	-	103 pcf
Max Preconsol Press	-	3500 psf



GRADING ANALYSIS

TWIN BUTTES MINE
Sample No. 22L



DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #1

Normal Stress - 2 ksf
Moisture Content - 8%
Dry Density - 157 pcf
Void Ratio - .11

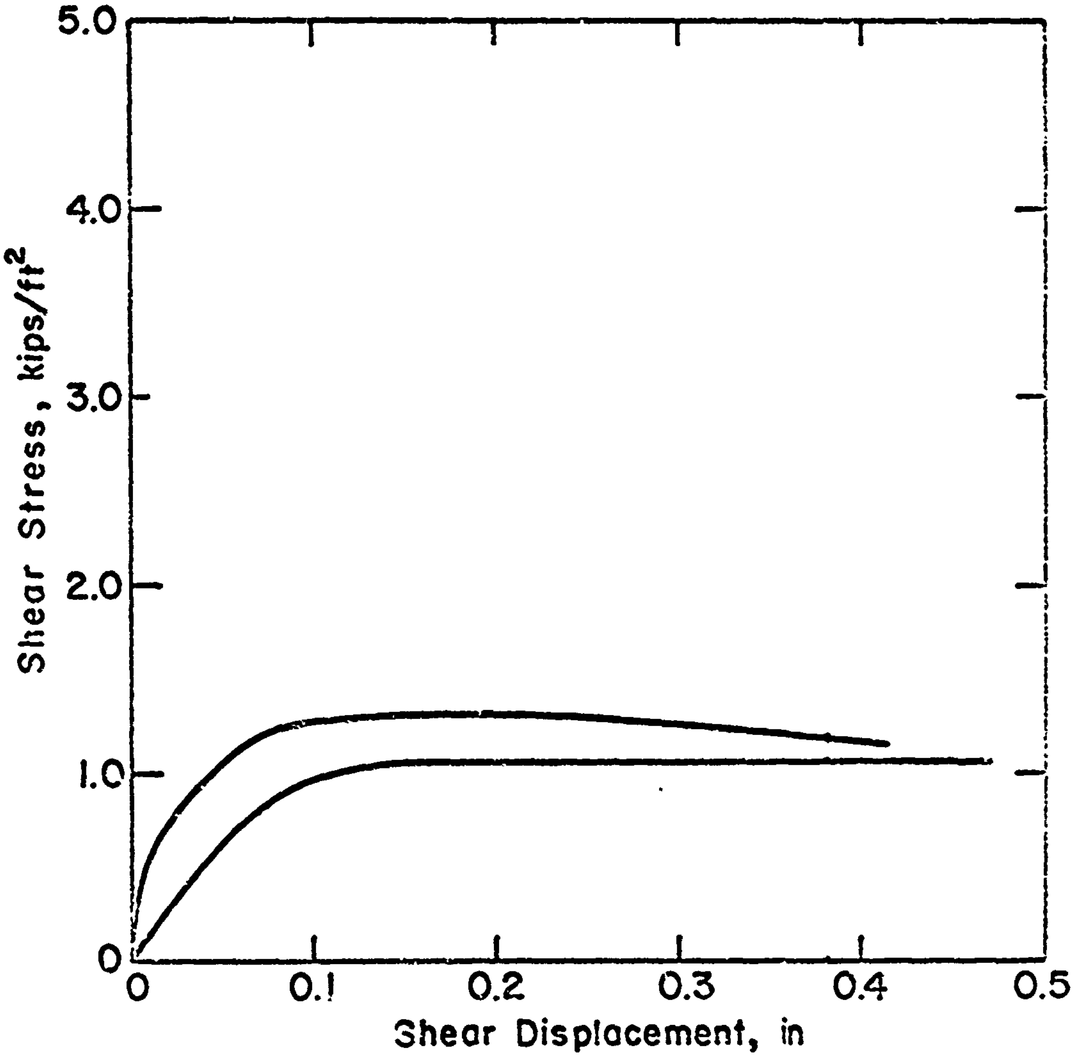
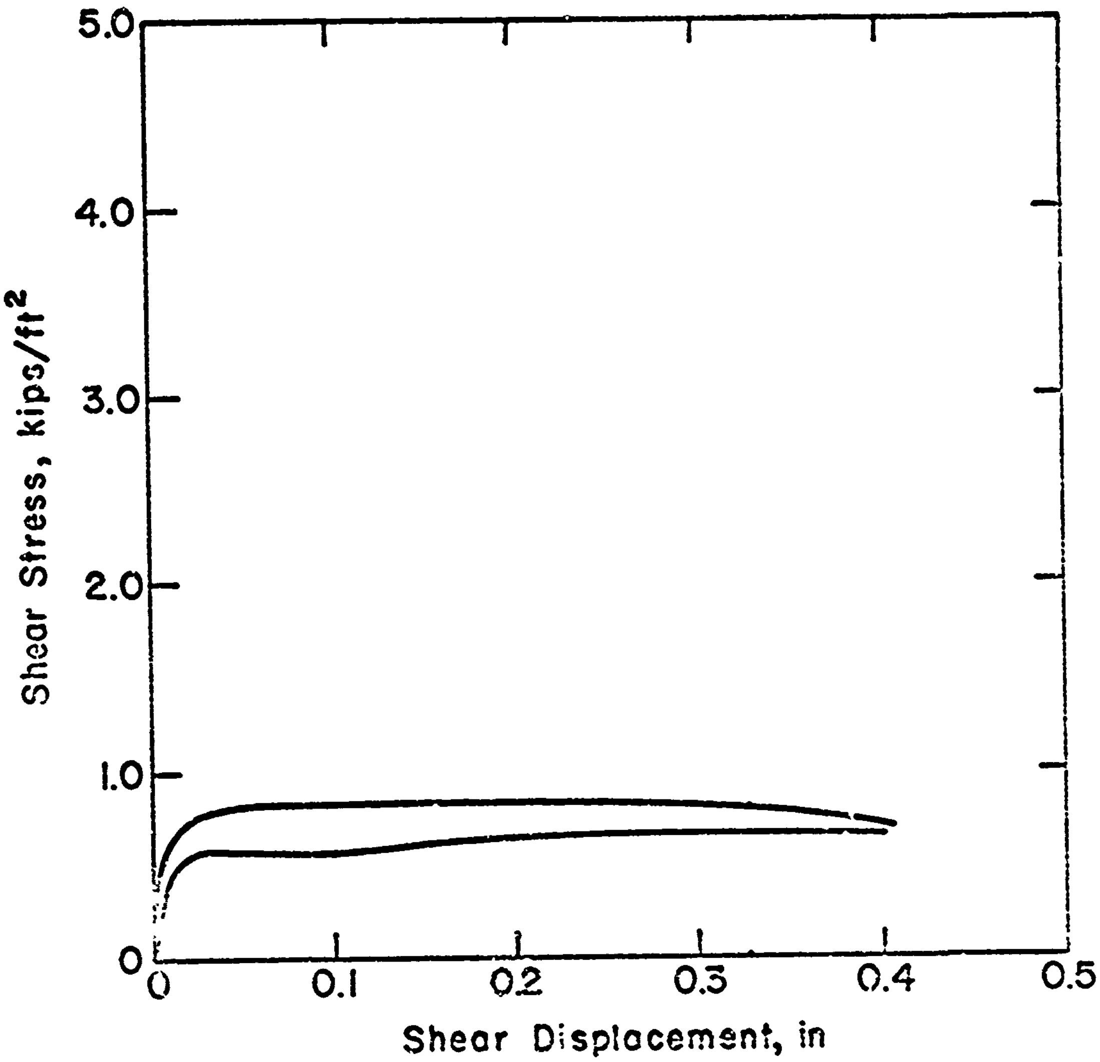


PLATE 13A

DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #2

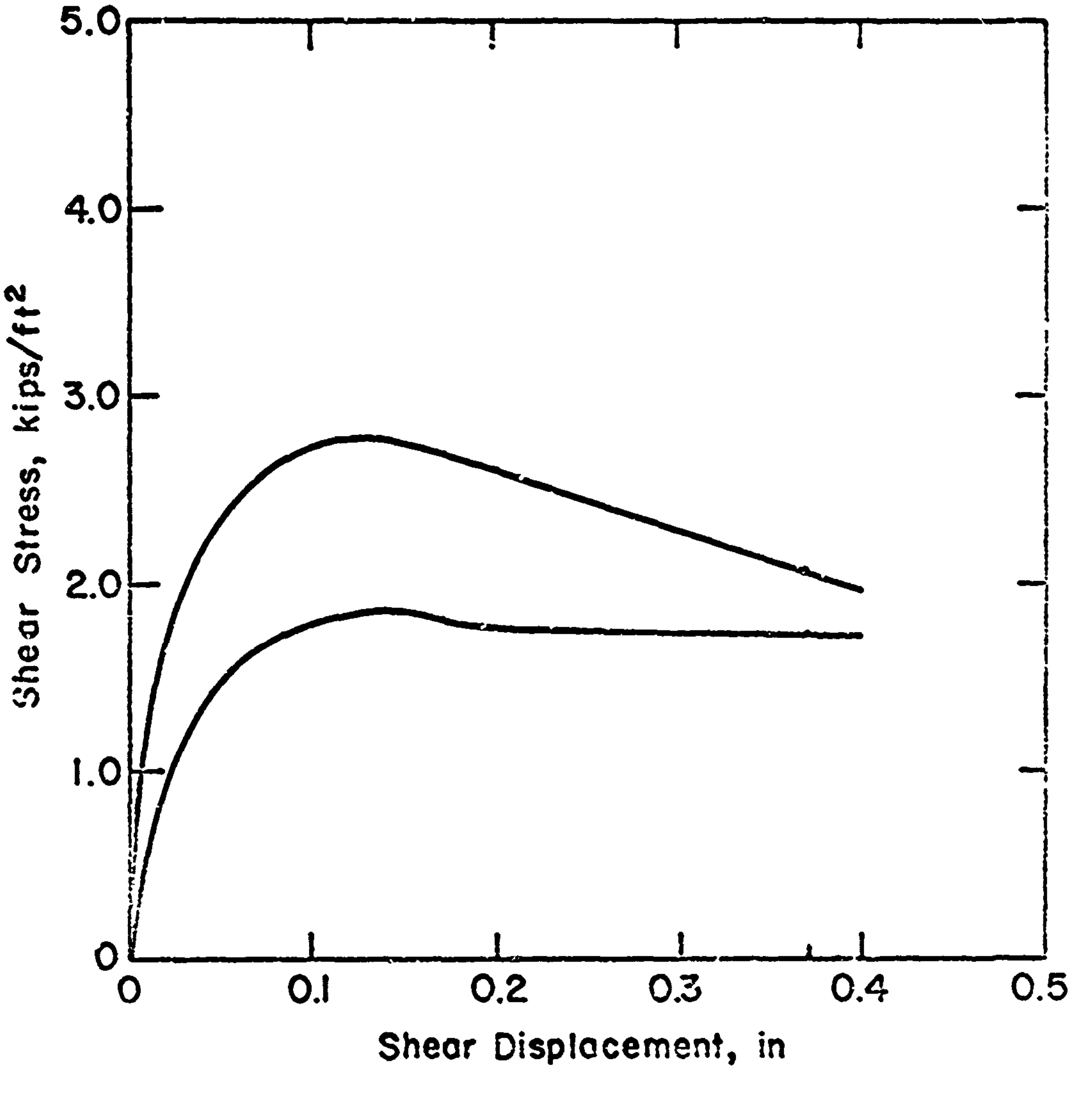
Normal Stress - 2 ksf
Moisture Content - 8%
Dry Density - 157 pcf
Void Ratio - .11



DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #3

Normal Stress	-	4 ksf
Moisture Content	-	6%
Dry Density	-	138 pcf
Void Ratio	-	.29



DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #4

Normal Stress - 4 ksf
Moisture Content - 8%
Dry Density - 135 pcf
Void Ratio - .29

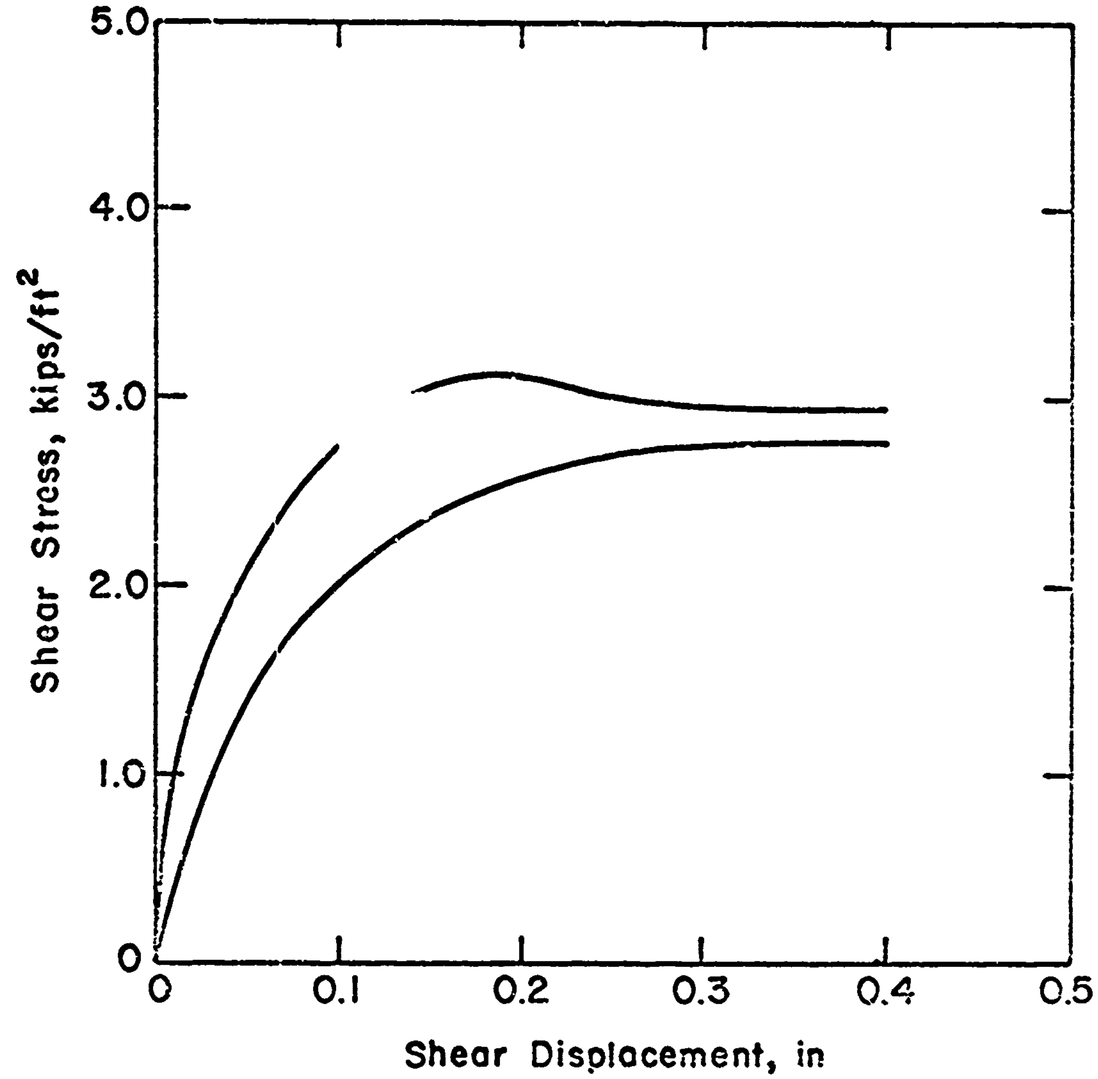


PLATE 16 A

DIRECT SHEAR TEST RESULTS

AUBURN DAM SITE
Sample No. F-1 #4

Normal Stress - 25 ksf
Moisture Content - 6.5%
Dry Density - 157 pcf
Void Ratio - .12

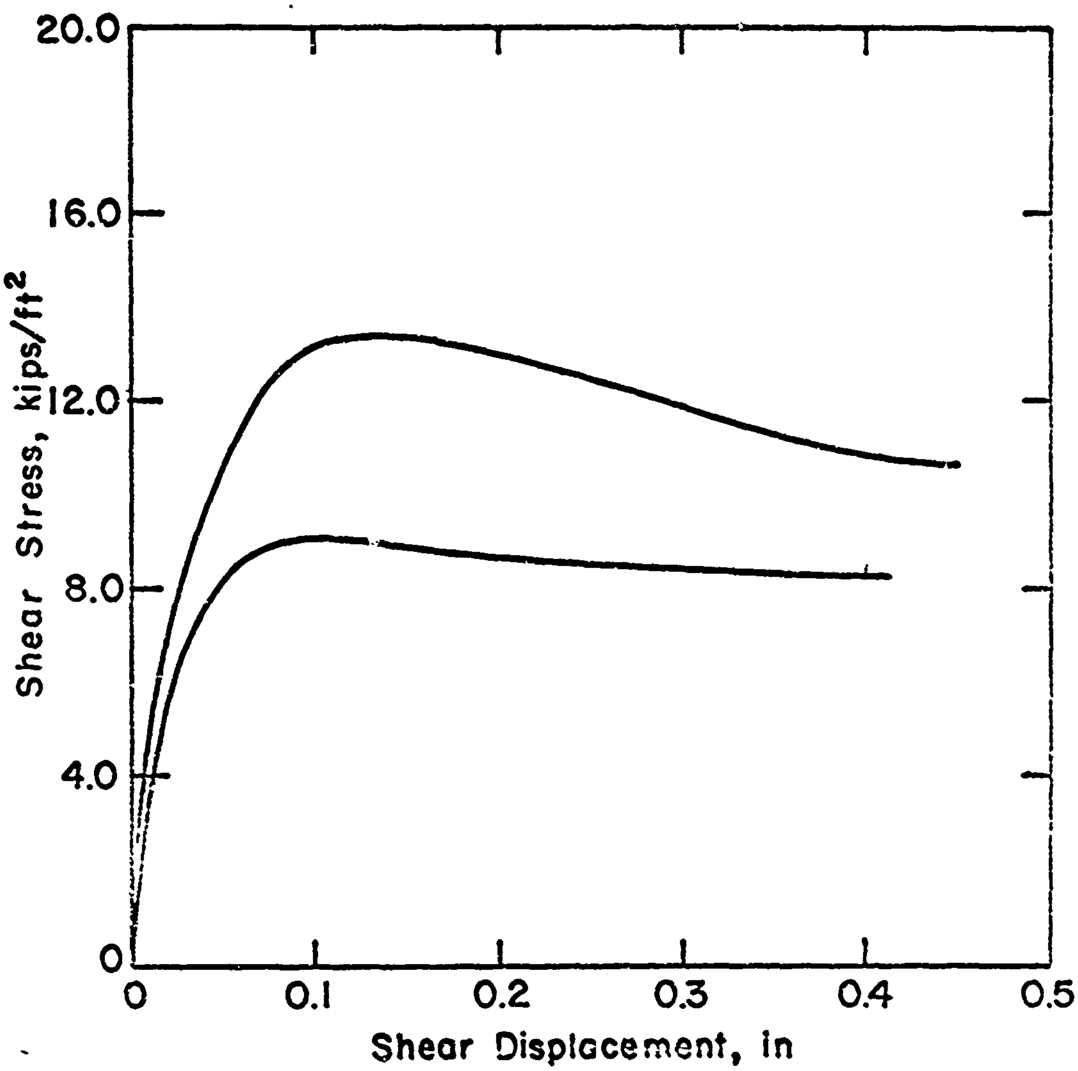
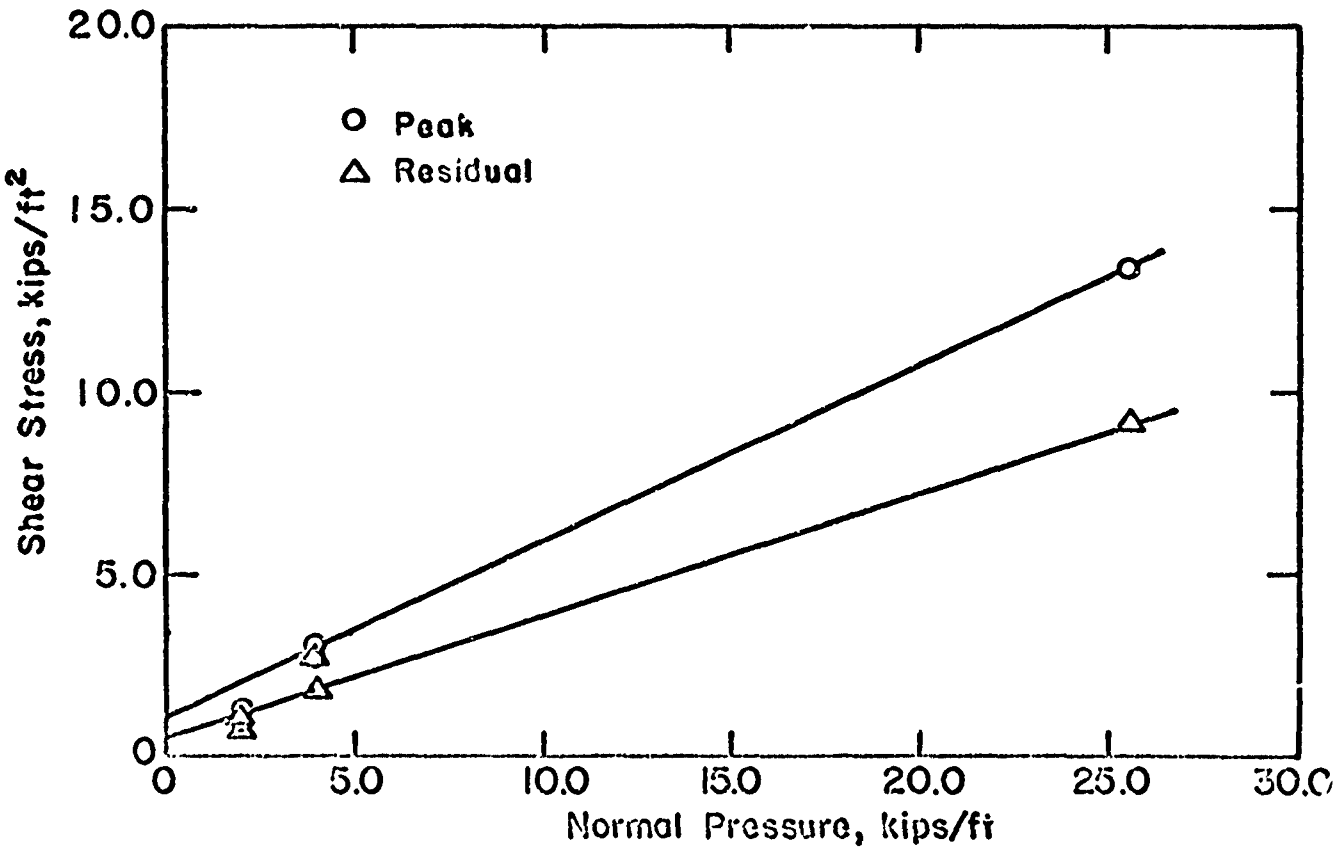


PLATE 17A

SHEAR TEST SUMMARY

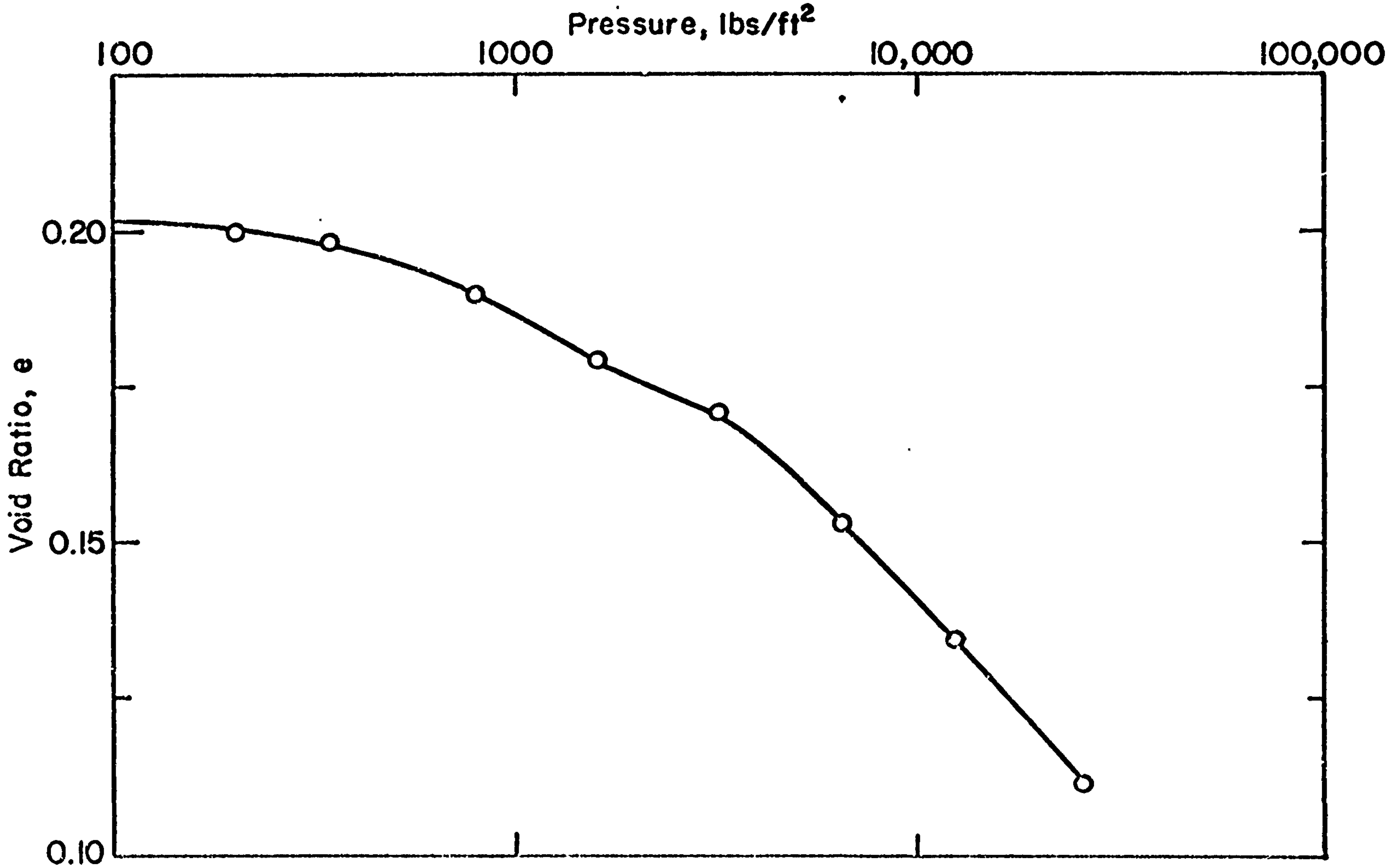
AUBURN DAM SITE
Sample No. F-1



AUBURN DAM SITE
Sample No. F-1

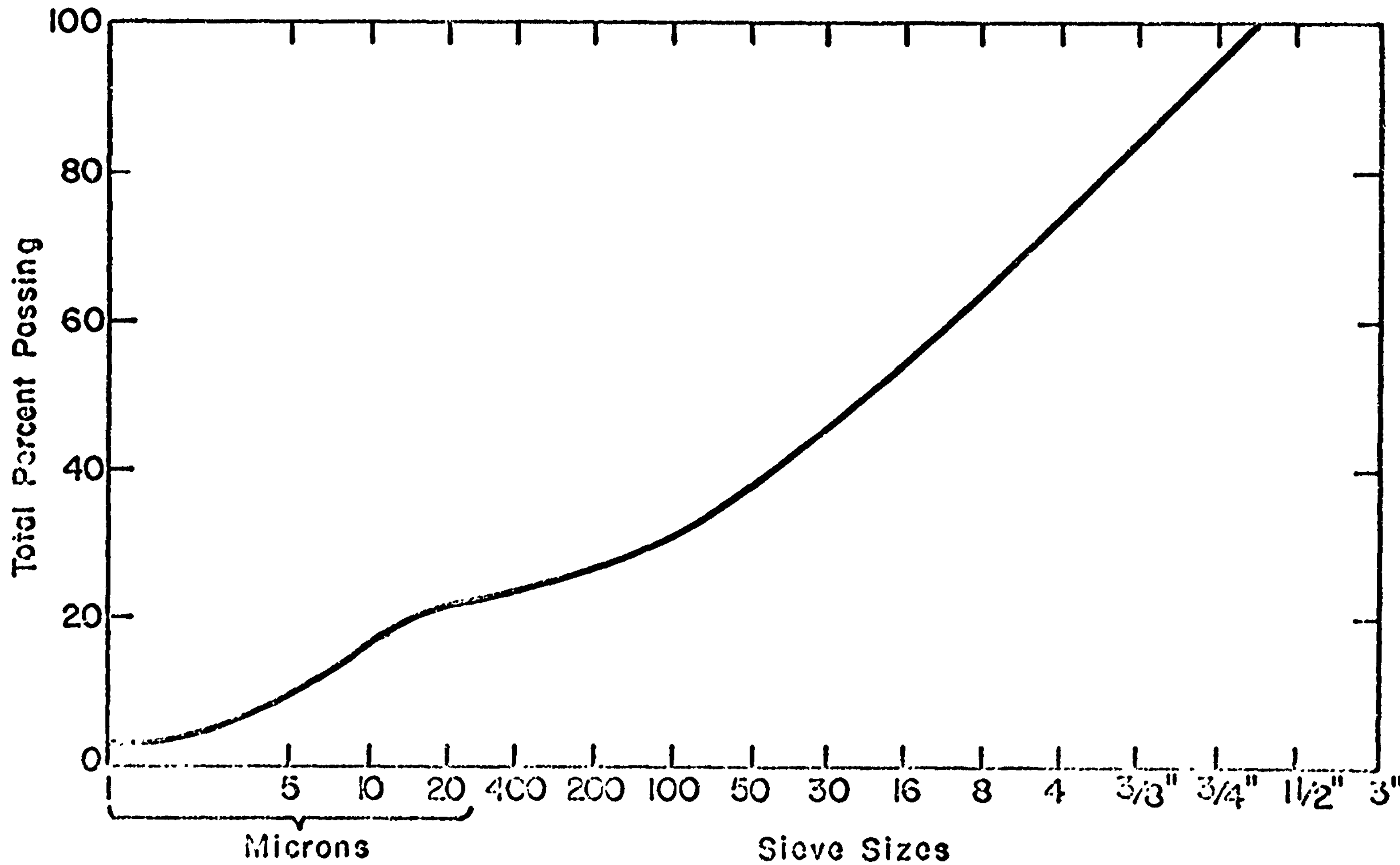
Atterberg Limits
LL - 19
PL - 19
PI - 0
A - 0

Specific Gravity - 2.80
Void Ratio - 0.21
Moisture Content - 7%
Dry Density - 157 pcf
Max Preconsol Press - 1500 psf



GRADING ANALYSIS

AUBURN DAM SITE
Sample No. F-1



APPENDIX B

Plate 1B

TWIN BUTTES MINE

Testing Summary

Sample	S/P	Sr/P	LL	PL	PI	-200
1	3.35	1.80	50	22	28	73
2	-	-	34	17	17	77
3	5.40	2.60	52	24	28	71
4	8.00	3.50	52	24	28	62
5	-	-	79	33	46	80
6	5.30	2.00	58	23	35	88
7	3.40	1.70	38	19	19	87
8	4.40	2.30	42	20	22	88
9	-	-	45	23	22	49
10	4.70	2.70	58	25	33	43
11	4.50	2.90	52	24	28	36
12	3.30	2.10	40	25	15	58
13	-	-	46	40	6	27
14	1.75	1.75	56	38	18	44
15	1.70	0.85	40	19	21	54
16	1.70	1.35	67	20	47	97
17	-	-	32	16	16	73
18	-	-	36	16	20	70
19	2.75	1.50	43	17	26	81
20	3.30	2.75	32	16	16	70

Plate 2B

TWIN BUTTES MINE

Mineralogy Study

Sample	Minerals	%	Sample	Minerals	%
1	Quartz	69	7	Quartz	65
	Montmorillonite	25		Feldspar	19
	Feldspar	6		Montmorillonite	12
	Illite	Trace		Illite	4
2	Quartz	85	8	Quartz	72
	$CaCO_3$	15		Feldspar	17
				Montmorillonite	11
				Illite	Trace
3	Quartz	49	9	Quartz	71
	Montmorillonite	43		Feldspar	15
	Illite	8		Montmorillonite	7
	Feldspar	Trace		Illite	7
4	Quartz	57	10	Quartz	80
	Montmorillonite	39		Feldspar	8
	Feldspar	4		Montmorillonite	6
				Illite	6
5	Quartz	65	11	Quartz	66
	Montmorillonite	19		Feldspar	17
	Illite	16		Montmorillonite	12
	Feldspar	Trace		Illite	5
6	Quartz	75	12	Quartz	59
	Montmorillonite	11		Feldspar	21
	Feldspar	10		Montmorillonite	12
	Illite	4		Illite	Trace